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DISCUSSIONS
APPLICATIONS FOR ADMISSION AND TRANSFER

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Founded November 5, 1852

PAPERS

WIND-BRACING CONNECTION EFFICIENCY

By U. T. Berg,[1] Assoc. M. Am. Soc. C. E.

Synopsis

This paper is intended as a warning, especially on wind connections in which negative moments due to gravity loads are neglected. Very high stresses in the connection members may be relatively harmless in themselves, but dangerous as far as the tension rivets are concerned, especially under alternate loading, where a gradual loosening may result. A condensed summary appears under "Conclusions" at the end of the paper.

Introduction

Wind resistance in structures is divided between resistance exerted by steel, walls, partitions, etc., in an unknown ratio. The mass inertia creates additional resistance against gusts of wind. The modern tendency toward increased height and relatively lighter weights intensifies the deflection and brings the question of rational design to the front.

What is the use of providing thousands of rivets that can not possibly do the work expected, and other rivets stressed to such a point that a few years of service will loosen them and increase the sway? To make architects "kneebrace-minded" would go far toward increased strength, as a conscientious design of the present-day types of connection meets many difficulties due to the large number of rivets required.

The term, "educated moments," is sometimes used in reinforced concrete design to denote that the reinforcement is placed more to suit the practical condition than the elastic theory can justify. Instead of providing for a support moment, $\frac{W L}{12}$, a smaller value, such as $\frac{W L}{16}$, or $\frac{W L}{24}$, is substituted, assuming that a re-adjustment of stresses will take place.

Tests substantiate the claim that, at working stresses and intermediate stages, re-adjustments take place only to a slight degree, but, at ultimate loads, re-adjustments become very marked.

NOTE.—Written discussion on this paper will be closed in April, 1932, *Proceedings*.

[1] Brooklyn, N. Y.

Fig. 1 illustrates the relations between computed and measured moments obtained from tests[2] on continuous beams. The conditions are that the beams are completely fixed at the ends: 47% of the bottom reinforcement steel is extended over the support. Steel designers generally take for granted that stresses will be distributed according to the connection; that the weaker parts will "give;" and that there is a balancing of stresses from the weaker to the stronger member. This practice may be justified within certain limits.

A beam with standard connections, as shown in Fig. 2, is designed as simply supported. If the column is unyielding, the connection angles must be sufficiently elastic to permit the beam ends to rotate. Angles, $\frac{3}{8}$ in. thick,

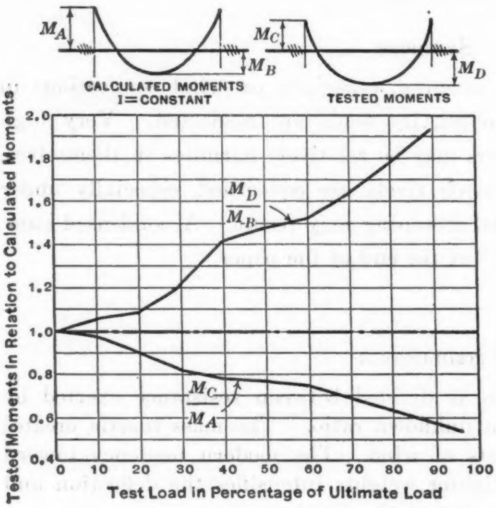

FIG. 1.

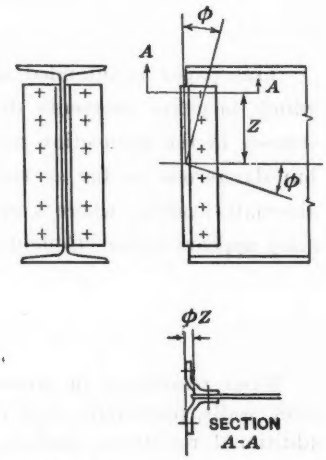

FIG. 2.

with 4-in. legs, will readily satisfy this requirement. The computed stresses in the angles are high, but harmless, because at no time is there any danger of excessive pull on the rivets. However, if the legs are made very thick the rivets, which are generally considered as taking shearing stresses only, will be called upon to take heavy tensile stress in addition, due to the increased strength of the connection.

Notation.—In addition to definitions given directly in the text, the notation in this paper is arranged for convenience in Appendix I.

Tension Rivets

It is logical to base the design on the stage in which the initial tension is reached on the rivet, and to express the factor of safety, as follows:

$$\text{Factor of safety} = \frac{\text{Initial tension value of rivet}}{\text{Total tension stress on rivet}}$$

[2] Deutschen Ausschuss für Eisenbeton, 41–47, "Versuche mit eingespannten Eisenbetonbalken," Technischen Hochschule zu Stuttgart, by Dr. Ing. C. Bach and O. Graf.

It is readily seen that the I-beam stub connection (Fig. 3 (*a*)), with a tension force of the value, $2S$, can not deform as illustrated in Fig. 3 (*b*). The initial tension in rivets would have to be overcome to permit the rivets to elongate. A force, $R - S$, is developed which maintains the edge in position as shown by Fig. 3 (*c*). When the initial tension is reached, it is

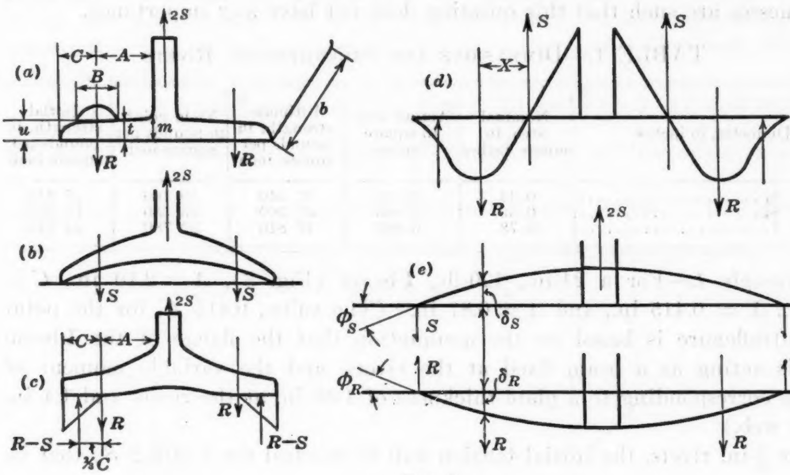

FIG. 3.

not unreasonable to assume that the force, $R - S$, is distributed as a triangular load, zero at the rivet center, and maximum at the edge. This would locate the center of gravity of the force at a distance equal to $\frac{2}{3}C$ from the rivet center. (Actually the pressure diagram is parabolic due to the deformation of the plate. However, the corresponding shorter arm is compensated for by the fact that the rotation due to the variation of pressure between the plates reduces the fixing moment at the rivet.)

If X denotes the distance from the point of contraflexure to the rivet center, the following equations can be written:

$$S X = (R - S) \tfrac{2}{3} C \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

or,

$$R = \frac{S(X + \frac{2}{3}C)}{\frac{2}{3}C} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2a)$$

$$S = \frac{\frac{2}{3}CR}{X + \frac{2}{3}C} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2b)$$

Reliable tests are available on tension rivets.[3] For red-hot driven rivets, the ultimate strength was found to be 54 000 lb. per sq. in.; the yield point, 37 000 lb. per sq. in.; and the initial stress at least 70% of the yield-point strength, or 25 900 lb. per sq. in. Table 1 gives the foregoing values applied to the rivets commonly used.

[3] *Bulletin No. 210*, Eng. Experiment Station, Univ. of Illinois, by Wilbur M. Wilson, M. Am. Soc. C. E., and William A. Oliver, Assoc. M. Am. Soc. C. E.

There is some element of doubt relative to the value, C, in Equation (1). It is possible that a reduced value of the distance from rivet center to the effective bearing edge should be entered in the equation when used for thinner plates, so that $C_{max.}$ is a function of the thickness of the plate plus one-half the diameter of the rivet head. However, in most practical cases, the plate thicknesses are such that this question does not have any importance.

TABLE 1.—DIMENSIONS AND STRENGTHS OF RIVETS

Diameter, in inches	Nominal area, in square inches	Actual area, in square inches	Ultimate strength, in pounds per square inch	Yield point, in pounds per square inch	Initial strength, in pounds per square inch
¾	0.44	0.518	27 540	19 160	13 410
⅞	0.60	0.690	37 260	25 530	17 870
1	0.78	0.886	47 840	32 780	22 940

Example 1.—For a 24-in., 120-lb., I-beam (Fig. 3), $A = 2.10$ in., $C = 1.5$ in., $X = 0.415$ in., and $A = 0.87$ in. (The value, 0.415 A, for the point of contraflexure is based on the assumption that the flange of the I-beam stub is acting as a beam fixed at the rivets, and the variable moment of inertia corresponding to a plate thickness of 1.06 in. at the rivets and 1.4 in. at the web.)

For $\frac{3}{4}$-in. rivets, the initial tension will be reached for a pull 2 S; that is,

$$2\,S = \frac{\frac{2}{3} \times 1.5 \times 13\,410 \times 2}{0.87 + \frac{2}{3} \times 1.5} = 2 \times 7\,170$$

which shows that the initial tension of a $\frac{3}{4}$-in. rivet equal to 13 410 lb. is overcome by the application of a force of only 7 170 lb. acting as described in this example.

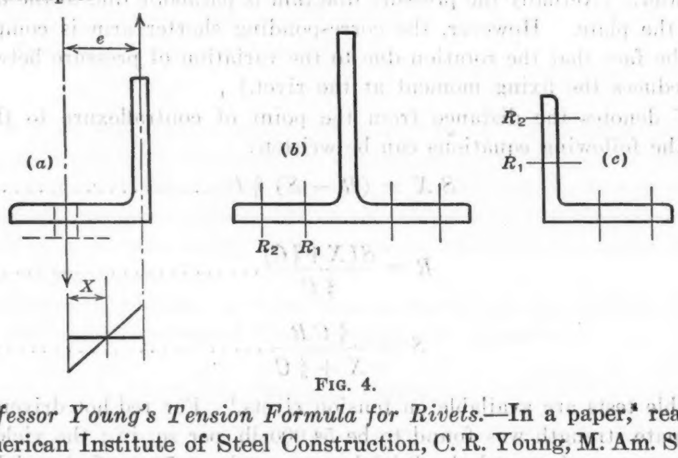

FIG. 4.

Professor Young's Tension Formula for Rivets.—In a paper,[4] read before the American Institute of Steel Construction, C. R. Young, M. Am. Soc. C. E., presented the following equation for tension stress in rivets:

$$f_r = 21\,000 - 8\,000\,d - 5\,500\,\sqrt{e} \quad (3)$$

in which, f_r is the permissible tension stress on the rivet (factor of safety

[4] *Engineering News-Record*, Vol. 100, 1928.

of 4); d, the diameter of the rivet, in inches, before driving; and, e, the eccentricity of loading on the rivet.

Equation (3) is based on tests using 4 by 4-in. angles. In comparing the values from it with the values derived from Equation (2a), it becomes important to use the right judgment relative to the eccentricity, e (Fig. 4(a)). If the test angles were free to move sidewise without any restraint on the outstanding leg, the values, e and X, should correspond. However, this condition is unlikely.

Assuming a condition as shown in Fig. 5 (a) and Fig. 5 (b), and expressed by $A - X = \frac{2 A^2}{4 A + B}$, the point of contraflexure of the angle would be located at a distance of about 0.65 e from the rivet center. (The foregoing

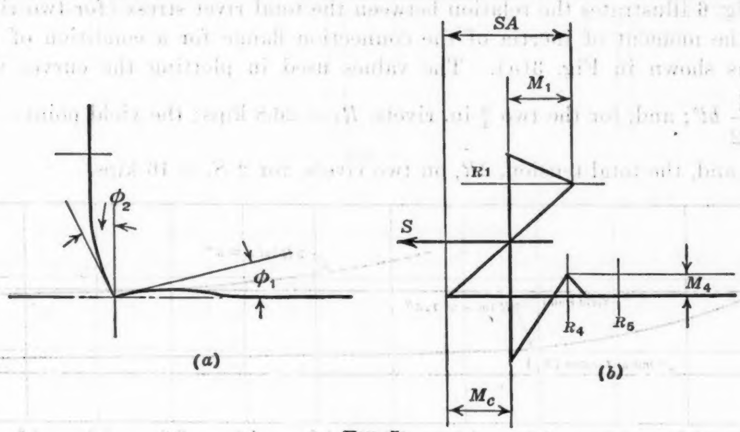

FIG. 5.

equation is derived subsequently.) Table 2 has been prepared giving permissible pull on the connection per rivet on the following assumptions:

For values from Equation (2a):

Factor of safety of 2 on initial stress on gross area of rivet, $C = 1.5$ in.

For values from Equation (3):

Factor of safety of 4 on ultimate strength, $e = \frac{X}{0.65}$.

TABLE 2.—COMPARISON OF WORKING STRESSES

Nominal size, in inches	X, in inches	Eccentricity, $e = \frac{X}{0.65}$, in inches	PERMISSIBLE TENSION PER RIVET, IN POUNDS: Equation (2b)	Equation (3)
$\frac{3}{4}$	0.75	1.15	3 840	4 000
$\frac{3}{4}$	1.50	2.31	2 690	2 930
$\frac{7}{8}$	0.75	1.15	5 100	4 850
$\frac{7}{8}$	1.50	2.31	3 560	3 400
1	0.75	1.15	6 500	5 500
1	1.50	2.31	4 600	3 630

Equation (1) applies only for stages at the initial tension of the rivet. Lower values are meaningless because the rivet stress is never less than the initial tension value. From this point on, the rivet elongation must be taken into account.

The following equation can be derived from the loading condition, as shown in Fig. 3(*e*):

$$\delta_S - \delta_R = \Delta_R = (R - R_i)\,\frac{g}{E\,F} \qquad (4)$$

in which, R is the total stress on the rivet; R_i, the total initial stress on the rivet; g, the rivet grip; F, the area of the rivet; E, the modulus of elasticity; Δ_R, rivet elongation; δ_R, rivet elongation due to the force, R; and δ_S, rivet elongation due to the force, S.

Fig. 6 illustrates the relation between the total rivet stress (for two rivets) and the moment of inertia of the connection flange for a condition of loading as shown in Fig. 3(*a*). The values used in plotting the curves were: $I = \frac{1}{12}\,b t^3$; and, for the two $\frac{3}{4}$-in. rivets, $R_i = 26.8$ kips; the yield point $= 38.3$ kips; and, the total tension, $2R$, on two rivets, for $2\,S, = 16$ kips.

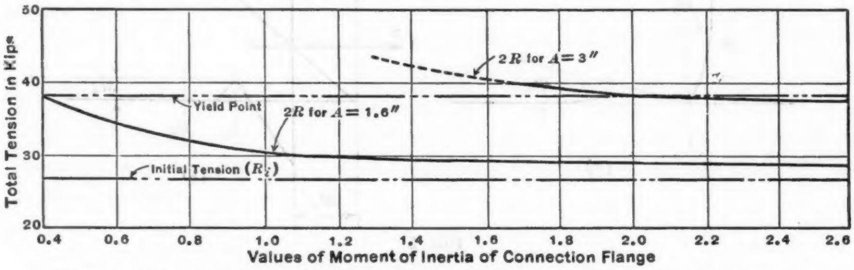

FIG. 6.—RELATION BETWEEN TOTAL RIVET STRESS AND MOMENT OF INERTIA OF CONNECTION FLANGE

For the sake of simplicity, the average thickness, t, was used instead of variations from u to m (Fig. 3(*a*)). From a study of Fig. 6, it is forcibly shown that the total tension developed on connection rivets under a constant force, $2\,S$, varies materially with the gauge distance, a, and with the thickness of the connection member up to a certain limit, after which increased thickness does not materially affect the tension developed.

Connections with more than one line of rivets, such as shown in Fig. 4, have merits only because of increased shearing value. Analyzed on the elastic theory, using commercial sizes, it is not possible to rely on any tension on the second line of rivets, R_2, unless the stress in the first line of rivets, R_1, exceeds not only the initial tension of the rivet, but also the yield point.

FLANGE ANGLE CONNECTIONS

A bending moment, M, at the joint, shown in Fig. 7 (*b*), is resisted by a couple, of force, S, acting on an arm, Y. The compression force, S, is transmitted directly into the column. The only bending effect at the lower angle is the influence of an eccentric action for the vertical load.

At the top, the tensile force would tend to pull the angle away from the face of the column, introducing bending moments in both legs of the angle. The elastic line of the deformed angle is unchanged at the rivet, R_1, which

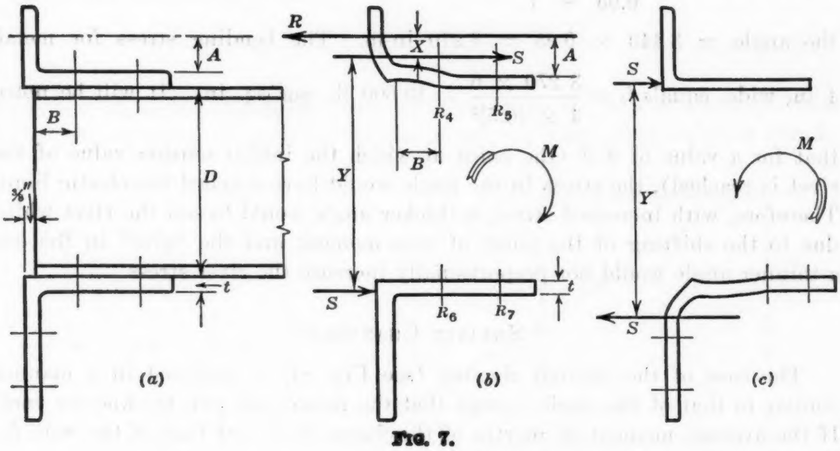

Fig. 7.

remains vertical, and at the rivet, R_4, which remains horizontal. Rivet R_5 has no other effect than horizontal shearing resistance.

Referring to Fig. 5, the angle is assumed to be completely fixed at R_4.

Hence, $M_4 = \frac{1}{2} M_c$; and, $\phi_1 = \frac{1}{4} M_c \dfrac{B}{E\,I} = \phi_2 = \dfrac{S\,A^2}{2EI} - \dfrac{M_c A}{EI}$.

The corner moment (M_c in Fig. 5(*b*)), is,

$$\frac{2\,S\,A^2}{4\,A + B} \qquad \text{(5)}$$

and the distance from the upper face of the angle to the point of zero moment equals,

$$(A - X) = \frac{2\,A^2}{4\,A + B} \qquad \text{(6)}$$

The total moment at the joint, M, equals $S\,Y$. Let the thickness of metal equal t, and the depth of the beam, D; then (Fig. 7),

$$Y = D + \frac{t}{2} + t + A - X \qquad \text{(7)}$$

For a moment in the opposite direction, the effect on the angles is reversed. The combined effect of bending on the lower angle from a force, S, plus eccentricity from the application of the direct load, would make a more serious condition than is the case for the top, except for the fact that, in most cases, the moment from the gravity loads is deductible from the wind moment, producing tension at the bottom.

Example 2.—For a top angle, 6 by $3\frac{1}{2}$ by $\frac{1}{2}$ in., $A = 2 - \frac{1}{2} = 1\frac{1}{2}$ in.; $B = 2\frac{1}{2} - \frac{3}{8} = 2.13$ in.; and, $C = 1.5$ in. From Equation (6), $A - X = 0.55$,

and $X = 0.95$ in. The permissible pull per rivet with a factor of safety of 2 is, $S = \frac{1}{2} \times \frac{13\,410}{0.95 + 1} = 3\,440$. The maximum bending moment on the angle $= 3\,440 \times 0.95 = 3\,270$ in-lb. The bending stress for metal, 4 in. wide, equals $f_s = \frac{3\,270 \times 6}{4 \times (0.5)^2} = 19\,600$ lb. per sq. in. It will be noted that for a value of 2 S (the point at which the initial tension value of the rivet is reached), the stress in the angle would have reached the elastic limit. Therefore, with increased stress, a thicker angle would loosen the rivet while, due to the shifting of the point of zero moment and the "give" in the leg, a thinner angle would not proportionally increase the rivet stress.

Sheared Channels

The case of the sheared channel (see Fig. 8), is analyzed in a manner similar to that of the angle, except that the flange and web thicknesses vary. If the average moment of inertia of the flange is I_F and that of the web, I_W, Equation (6) can be re-written for channels:

$$(A - X) = \frac{A^2}{2A + \frac{I_F}{I_W} \times \frac{B}{2}} \quad \text{.......................(8)}$$

Sheared I-Beams

This case has been covered under the section, "Tension Rivets." Note that $Y = D + t$. For a constant thickness, t, of the flange, $X = \frac{1}{2} A$, and for a straight-line variation from u (the thickness at the rivet) to m (the thickness at the web), the distance, X, is equal approximately to,

$$X = \frac{u}{u + m} A \quad \text{.......................(9)}$$

Behavior Beyond the Elastic Limit

The equations developed for the location of points of contraflexure of connections have been based on stresses within the elastic limit. Above the elastic limit, re-adjustments take place. The writer is not in a position definitely to determine these re-adjustments. Probably tests would be required. However, some approximate values can be derived. A redistribution of the moments in relation to the section modulus does not seem unreasonable. In that case, the distance, X, equals, approximately,

$$X = \frac{u^2}{u^2 + m^2} A \quad \text{.......................(10)}$$

The same reasoning applied to angles would make $X = \frac{1}{2} A$ for all cases above the elastic limit.

Rotation from Deformation of Connections

The rotation of the beam end, due to deformation of the connection (see Fig. 9), can be expressed:

$$\phi = \tan \phi = \frac{(\Delta_1 + \Delta_2 + \Delta_3 + \Delta_4)\, S}{D + t} \quad \ldots\ldots\ldots\ldots\ldots (11)$$

in which, Δ_1 is the elongation due to bending in the top connection from $S = 1$; Δ_2, the elongation due to tension in the top connection only, for $S = 1$;

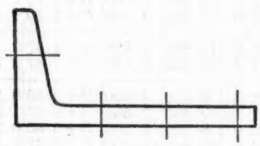

FIG. 8.

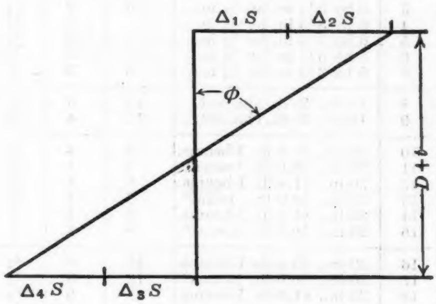

FIG. 9.

Δ_3, the shortening in the bottom connection due to $S = 1$; and, Δ_4, the shortening due to compression when $S = 1$.

For a case as shown in Fig. 7(*b*):

$$\Delta_1 = \frac{1}{3\, E\, I_1}\left[X^3 + (A - X)^3 + (A - X)^2 B\, \tfrac{3}{4} \right] \ldots\ldots\ldots (12)$$

and,

$$\Delta_2 = \frac{l}{E\, F'} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which, l is the effective length of connection, and F', the total area of web for sheared **I**-beams or horizontal leg for angles. Δ_3 = shortening in the bottom connection due to bending from $S = 1$. In most cases this value is zero, as the connection butts up against the column or adjacent connection. Δ_4 = shortening due to compression from $S = 1$. For most cases $\Delta_4 = \Delta_2$.

For a total moment, M, at the joint, ϕ is the angle of rotation due to deformation (see Fig. 9), and can be expressed: $\phi = k\, M$, or,

$$k = \frac{\phi}{M} = \frac{\Sigma\, \Delta\, S}{(D + t)\, Y\, S} = \frac{\Sigma\, \Delta}{(D + t)\, Y} \quad \ldots\ldots\ldots\ldots (14)$$

Table 3 gives values of Δ_1 and Δ_2, when S equals unity, for typical commercial connections. This table also gives the total pull, S, which stresses rivets up to 75% of the initial tension value, and the corresponding stress on flange and web. In Column (1), the various types correspond with those shown in Fig. 10.

TABLE 3.—CHARACTERISTICS OF TYPICAL COMMERCIAL CONNECTIONS

Type of member (see Fig. 10.)	Item No.	Size	Length, b, in inches	Tension Rivets		Total pull on connection, in kips	Unit Stress, f, in Pounds per Square Inch		Lever arm, y, in inches	Elongation Factor	
				Number	Diameter, in inches		f_F	f_W		EA_1	EA_2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
	1	4 by 3 in. by $\frac{5}{16}$ in.....	6	2	$\frac{3}{4}$	5.3	37 000	2 800	....	.....	....
	2	4 by 3 in. by $\frac{1}{2}$ in.....	6	2	$\frac{3}{4}$	10.3	33 400	3 400	$D+1.42$	13.2	0.85
B	3	6 by $3\frac{1}{2}$ in. by $\frac{1}{2}$ in...	6	2	$\frac{3}{4}$	7.1	27 000	2 340	$D+1.05$	8.40	1.50
	4	6 by $3\frac{1}{2}$ in. by $\frac{5}{8}$ in...	.....	2	$\frac{3}{4}$	10.7	24 000	2 880	$D+1.11$	3.26	1.20
	5	6 by $3\frac{1}{2}$ in. by $\frac{3}{4}$ in...	6	2	$\frac{3}{4}$	11.1	16 000	2 460	$D+1.19$	1.46	1.00
	6	6 by $3\frac{1}{2}$ in. by $\frac{3}{4}$ in...	.....	2	$\frac{7}{8}$	14.8	21 400	3 300	$D+1.19$	1.46	1.00
	7	6 by $3\frac{1}{2}$ in. by $\frac{3}{4}$ in...	6	2	1	19.0	27 400	4 200	$D+1.19$	1.46	1.00
C	8	15-in., 50-lb. channel...	12	4	$\frac{3}{4}$	18.0	14 800	2 120	$D+1.47$	2.50	0.70
	9	15-in., 50-lb. channel...	12	4	$\frac{7}{8}$	24.2	20 000	2 800	$D+1.47$	2.50	0.70
D	10	20-in., 81.4-lb. I-beams.	8	4	$\frac{3}{4}$	22.4	6 550	4 650	$D+0.60$	0.24	1.25
	11	20-in., 56.0-lb. beams*.	8	4	$\frac{3}{4}$	22.50	19 400	7 500	$D+0.37$	1.25	2.00
	12	20-in., 81.4-lb. I-beams.	8	4	$\frac{7}{8}$	30.0	8 800	6 200	$D+0.60$	0.24	1.25
	13	20-in., 56.0-lb. beam*.	8	4	$\frac{7}{8}$	30.0	25 800	10 000	$D+0.37$	1.25	2.00
	14	20-in., 81.4-lb. I-beams.	8	4	1	38.4	11 250	8 000	$D+0.60$	0.24	1.25
	15	20-in., 56.0-lb. beams*.	8	4	1	38.50	33 000	12 800	$D+0.37$	1.25	2.00
E	16	20-in., 81.4-lb I-beams.	12	8	$\frac{3}{4}$	44.8	8 700	6 250	$D+0.60$	0.16	1.04
	17	20-in., 56.0-lb. beams*.	12	8	$\frac{3}{4}$	45.00	25 800	10 000	$D+0.37$	0.84	1.78
	18	20-in., 81.4-lb. I-beams.	12	8	$\frac{7}{8}$	60.00	11 700	8 320	$D+0.60$	0.16	1.04
	19	20-in., 56.0-lb. beam*.	12	8	$\frac{7}{8}$	60.00	34 200	13 300	$D+0.37$	0.84	1.78
	20	20-in., 81.4-lb. I-beams.	12	8	1	76.8	15 000	10 700	$D+0.60$	0.16	1.04
	21	20-in., 56.0-lb. beam*.	12	8	1	77.0	44 000	17 000	$D+0.37$	0.84	1.78

* Bethlehem Section; C (assumed) = $\frac{1}{2}$ rivet head + $\frac{3}{2}$ thickness, u, at the rivet.

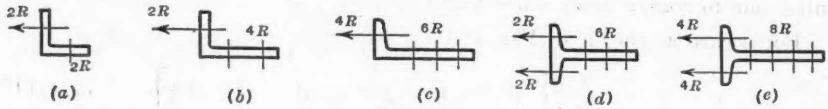

FIG. 10.

JOINTS

Gravity Loads Only.—For joint equilibrium the following rotations must balance:

(*a*) The rotation due to a gravity load on a freely supported span is,

$$\phi_{GA} = \frac{W L^3}{24 E I} \quad \text{.........................(15)}$$

(*b*) The rotation due to negative moments induced by gravity loads is,

$$\phi_{MA} = \frac{L}{E I}\left[\frac{M_{GA}}{3} + \frac{M_{GB}}{6}\right] \quad \text{..................(16)}$$

and, for a typical symmetrical case, in which, $M_{GA} = M_{GB} = M_G$,

$$\phi_{MA} = \frac{L}{2 E I} M_G \quad \text{....................(17)}$$

(*c*) The rotation due to the connection is, $\phi_k = k\, M_{GA}$; thus,

$$\frac{W L^3}{24 E I} = \frac{L}{2 E I} M_G + k\, M_G \quad \text{............(18)}$$

and,

$$M_G = \frac{1}{L + 2 E I k} \times \frac{W L^3}{12} \quad \text{................(18a)}$$

Wind Forces Only.—For wind forces, as shown in Fig. 11(*b*), and for the corresponding moments, M_{HA} and M_{HB}, the column rotation can be expressed:

$$\phi_{HA} = \frac{L}{6\,E\,I}\,(2\;M_{HA} + M_{HB}) \;\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

from beam deformation, and,

$$\phi_{HA} = k\;M_{HA} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

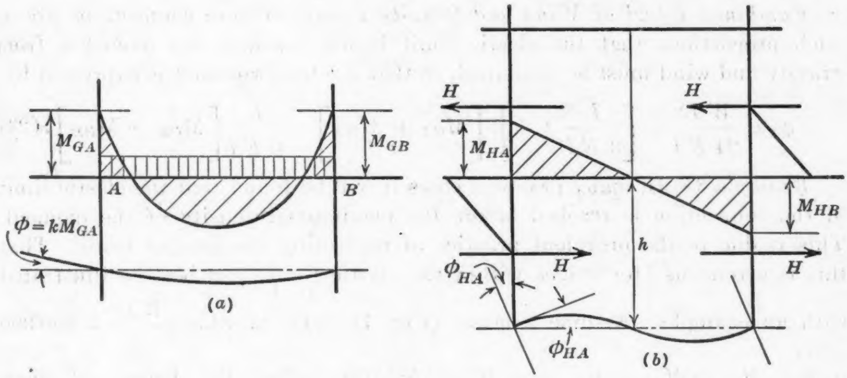

FIG. 11.

from the "give" in the connection (see Equation (14)). If $M_{HB} = -\,M_{HA}$, the total rotation due to wind is,

$$\phi_{HA} = M_{HA}\left(\frac{L}{6\,E\,I} + k\right) \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Equation (21) expresses the rotation of the joint due to the wind forces, including the effect of the "give" in the connection. This rotation, it should be noted, also expresses the unit horizontal deflection, as the influence from

the gravity loads is small and can be neglected, so that the horizontal deflection from one story to the next can be assumed equal to:

$$q\,\phi_{HA}\,h + \frac{H \cdot h^3}{12\,E\,I_c} \qquad \text{.......................(22)}$$

in which, h is the story height; H, the wind force (Fig 11(b)); and I_c, the moment of inertia of the column.

Combined Effect of Wind and Gravity Loads.—Where connections are of such proportions that the elastic limit is not reached, the moments from gravity and wind must be combined, so that the total rotation is expressed by:

$$\phi_A = \frac{W\,L^3}{24\,E\,I} - \left[\frac{L}{3\,E\,I} + k\right]\left[M_{GA} + M_{HA}\right] - \frac{L}{6\,E\,I}\left[M_{GB} - M_{HB}\right]. \quad (23)$$

Example 3.—In many practical cases it will be found that the elastic limit of the connection is reached before the maximum intensity of the moment. This is due to the prevalent practice of neglecting the gravity loads. That this is erroneous (for stages within the elastic limit) can best be illustrated with an example. Assume a beam (Fig. 11 (e)), in which, $\frac{W L^2}{8} = 900\,000$ in-lb.; $M_{GA} = M_{GB} = M_{HA} = -M_{HB} = 585\,000$ in-lb.; the length of span $= 20$ ft.; the depth of beam $= 12$ in.; $I = 200$; and $\Delta_1 + \Delta_2 + \Delta_3 + \Delta_4 = \frac{2}{E}$.

The total work done in deforming all fibers, is, for beams:

$$W_1 = \tfrac{1}{2} \sum \frac{M^2\,\Delta_x}{E\,I} \qquad \text{.......................(24)}$$

and for connection,

$$W_2 = \tfrac{1}{2}\left(\sum \Delta\right)\frac{M^2}{Y^2} \qquad \text{.....................(25)}$$

This gives a total of 3 830 in-lb. for a condition equivalent to the moment curve, $G + H$, and 10 940 in-lb. for the moment curve, H, or nearly three times as much work. Therefore, from the theory of least work, the curve, $G + H$ (combined gravity and wind), is the only true moment curve.

Since the floor acts as a monolithic mass, the side movement (deflection) as expressed by Equation (22), must be the same for all joints. A weaker joint is relieved by the stronger, to some extent compensating for any misjudgment by the designer.

Relative Behavior When Connection is Stressed to the Elastic Limit

The connection may reach the elastic limit in one of the following ways:

(A) The web of the sheared I-beam, or the horizontal leg of the angle, reaches the elastic limit from tension.

(B) The flange of the sheared I-beam, or the vertical leg of the angle, reaches the elastic limit from bending.

(C) The rivet stress reaches the elastic limit.

If it is not possible to design the connection within the elastic limit, the writer believes that care should be taken at least to protect the rivets from being overstressed.

A study of the relative stresses as shown Table 3 will help to balance the design. For example, for flange-angle connection, the rivets are relatively stronger than the vertical leg up to and including, angles, $\frac{5}{8}$ in. thick, for $\frac{3}{4}$-in. rivets, and angles, $\frac{3}{4}$ in. thick, for $\frac{7}{8}$-in. rivets. Many of the sheared I-beam connections as used to-day would reach the elastic limit on the rivets long before the flange and web stresses are nearly that high.

Bending Moment Effects

Consider for a moment gravity loads alone. Comparing the bending moment as computed from Equation (18), with the rigid frame (in which, $k = 0$), it will be found that the connection does not lower, to any great extent, the value of the negative moment. The effect of an angle connection may be as great as 10% reduction, while the effect of the sheared I-beam is nearer 4 per cent. Therefore, the moments as obtained from monolithic frame action are from 4 to 10%, and even more, on the safe side.

Designers should not neglect checking the connection for gravity loads only. Especially for the upper stories, under relatively smaller wind forces, the negative moments induced from gravity loads will predominate. These gravity negative moments are serious because they exist on both sides of the joint (Fig. 11(f)), with approximately the same intensity, and there is no balancing effect from the weaker to the stronger connection; while in a wind moment, if the elastic limit is reached on one side of the column, the bottom connection on the opposite side of the same column (Fig. 11(g)), will immediately take its capacity of the remainder.

Fig. 11(d) illustrates the case in which the gravity loads do not carry the stresses to the elastic limit. Not all the wind moment, M_{HA}, but a part of it, can be added before the limit is reached. A redistribution takes place whereby the remainder of the wind moment is taken by the connection on the opposite side of the column. (The wall column is an exception. In this case the monolithic floor will readjust the stresses, transferring them to the interior joints.)

If the wind moments are greater than the gravity moments, the main objection to the common method of using wind moments only in designing the connection, is that the elastic limit is likely to be reached on some part of the connection and harmful effects may arise, particularly the danger of loose rivets under repeated stress.

Summary and Conclusions

Formulas have been developed for the determination of tension in rivets, effective lever arm of connection, rotation of joint, joint equilibrium, horizontal deflection (sway), etc., for the commonly used connections, such as flange angles, sheared channels, and sheared I-beams. The insecurity in the

present method of rivet design has been illustrated, showing particularly how a stress of 18 000 lb. per sq. in. on the rivet as used by many designers, may develop stresses actually twice that amount.

The importance of analyzing the relative strength of the various parts of the connection has been stressed, and a warning against making the rivets the weakest link has been given. Finally, the designer has been cautioned against neglecting gravity loads, particularly under the condition when the wind forces are comparatively small.

Discrepancies between current practice and rational analysis emphasize the need for further experimentations along these lines.

With the introduction of lighter forms of construction, making buildings more vulnerable to wind, the influence of riveted connections on the life of structures becomes increasingly important, and should be reflected in better balanced details.

APPENDIX I

Notation

In the paper, the following notation is adopted:

a = gauge distance.
b = length of a structural element in a connection.
d = diameter of rivet before driving.
e = eccentricity of a load acting on a rivet.
f = unit stress; f_s, due to bending; f_F, stress in the flange, or in the vertical leg of an angle; and f_W, stress in the web, or in the horizontal leg of an angle; f_r, stress in rivet.
g = rivet grip.
h = story height.
k = a coefficient in Equation (14) $= \frac{\phi}{M}$.
l = effective length of a connection.
m = thickness of a flange at the face of the web.
q = horizontal deflection.
t = average thickness of a flange between the center of a rivet and the face of the web.
u = thickness of the flange at the center line of the rivet.
w = work (w_1) in beams, or (w_2) in connections.
z = a polar ordinate (see Fig. 2).
A = distance from face of web to center line of rivet.
B = diameter of rivet head.
C = distance from center line of rivet to effective bearing edge of flange.
D = depth of beam.
E = modulus of elasticity.
F = area of a rivet; F' = total area of web for sheared I-beams, or horizontal leg for angles.
H = a wind force.
I = moment of inertia; I_F = average for the flange; I_W = average for the web; and I_c = average for the column.
L = span length.

M = bending moment. (A subscript, c, denotes moment in the corner of an angle; Subscripts g and h denote moments due to gravity and wind loads, respectively; and Subscripts A and B denote moments at the ends, A and B, respectively, of a beam.)

R = a tensile force on a rivet. (A subscript, i, denotes initial force.)

S = an eccentric force opposing R.

W = total weight.

X = distance from the point of contraflexure to the center line of rivet.

Y = a moment arm.

δ = the elongation of a rivet; δ_S, due to the force, S; and δ_R, due to the force, R.

Δ = total elongation, or shortening, of a rivet. Numerical subscripts denote cause of elongation, as follows: (1) Bending in the top connection due to $S = 1$; (2) tension in the top connection due to $S = 1$; (3) bending under $S = 1$, causing shortening in bottom connection; and (4) compression under $S = 1$, causing shortening.

ϕ = rotation of the end of a beam due to the deformation of the section, ϕ_K, due to the connection; and ϕ_M, due to bending moments induced by gravity loads. The significance of the subscripts, G, H, A, and B, is as defined under M.

[illegible]

[illegible] of a beam)

[illegible]

[illegible] the deformation of the [illegible] due to bending [illegible]. The distributions of the [illegible]

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

PUBLIC SUPERVISION OF DAMS[1]

A SYMPOSIUM

NOTE.—Written discussion on this Symposium will be closed in April, 1932, *Proceedings*.

[1] Papers presented at the meeting of the Power Division, New York, N. Y., January 16, 1930, and at the Technical Session, Sacramento, Calif., April 23, 1930, respectively.

RECOMMENDATIONS FOR LEGISLATION AND FOR APPLICATION OF LAW

By A. H. Markwart,[2] M. Am. Soc. C. E.

Synopsis

Provided that State supervision of dams is to obtain, this paper contains recommendations upon the means to be taken for the regulation of design, construction, and maintenance. Characteristics of the best types, the extent, exemptions, personnel, and costs of supervision are discussed, and the paper concludes with comments on the feasibility of building codes for dams.

Popular Interest in Dams

The failure of an engineering structure, unless attended by loss of life or property, does not usually attract the attention of the layman. When such a failure results in damage to property only, it is of interest to the owner or owners of the structure and to the party or parties damaged. When, however, the failure involves loss of life as well as damage to property, it becomes a tragedy which is the subject of great concern to the general public.

There are probably no engineering structures which the public fears more than dams. While the layman, as a rule, cannot visualize the effects of failure of many engineering works, the destruction which would follow the failure of a dam is to him most apparent. For this reason, there is perhaps more popular interest in dams than in other engineering construction.

The failure of the St. Francis Dam in Southern California was a tragic occurrence which has prompted many to question seriously the stability of existing dams. It does not follow, however, that the building of dams must be discontinued any more than that mining activities should be discontinued because of loss of life in coal-mining operations.

The conservation of fuel resources by the building of storage works for power purposes must be continued. The irrigation of lands from storage resources must also be continued, and flood-control reservoirs must be built, unless other and better means are devised for handling flood flows. What the failure of the St. Francis Dam and other ill-fated dams teaches, is the necessity of introducing greater conservation and sounder judgment, particularly the latter, in the design and construction of such structures.

Necessity for Supervision

It is being realized more and more, as intelligent investigations of dam failures are made, that the critical defects have been almost invariably avoidable, had proper judgment and foresight been exercised in the design and construction. Such avoidable defects can be eliminated almost entirely with effective regulation and control.

[2] Vice-Pres., Pacific Gas & Elec. Co., San Francisco, Calif.

In exerting supervision over dams, the State is merely exercising one of its police functions in the interest of the public. State supervision of dams is desirable if properly done, and undesirable if not properly done since it then gives a false security to those who place their trust in its protection.

State interference in private matters is always to be deplored, but protection to life, health, and property is in the public interest. It is regrettable that, where these are endangered, resort to Government is invariably had to effect correction. The reform movement which always follows finally centers in further governmental pressure restricting the individual or local group. The same result might have been achieved by action from within the group, if those in responsible positions would voluntarily exercise the necessary control over dangerous practices. Individuals, corporations, and local governments should know of themselves what is essential for the protection of life and property and should impose the necessary self-discipline to eliminate the risks. It is apparent, however, with human nature constituted as it is, that this control from within is not to be expected.

The public has a right to the protection of life and property from the possible failure of dams which impound large volumes of water, and in the absence of self-imposed regulation, it is the duty of Government to provide legislation to that end. This will not necessarily provide a perfect assurance of safety, however, because the criticism may be made that governmental agencies are no more infallible than private agencies, and that all the engineers responsible for the design of dams which have failed, would have been qualified to act as consulting engineers in the service of the State. The value of State supervision does not consist in requiring of the owner the employment of a consulting engineer, but rather in requiring an independent review of the design, insuring the application of more than one mind to the problem. In the words of the findings on the St. Francis Dam disaster, "the construction and operation of a great dam should never be left to the sole judgment of one man, no matter how eminent."[3]

LEGISLATION

Considerable legislation has been enacted in the United States, most of it conservative and well considered. The States of Colorado, Idaho, Massachusetts, New Jersey, New York, Pennsylvania, and Washington have statutes bearing on this subject, some of them of long standing. California enacted a statute, to take effect July 1, 1929.

Other Countries.—All the more important foreign countries have some form of regulations looking toward the safety of dams. These, however, differ fundamentally from those in force in the United States, in that they stipulate the theoretical design methods and the working stresses to be used, rather than limit their authority to the approval of the work of others.

France, since 1897, has had in force, regulations applicable to gravity dams and is formulating similar regulations for other types of dams. Great

[3] Verdict filed by Jury Drawn by the Coroner of Los Angeles County to Consider Testimony on St. Francis Dam Failure, *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2162.

Britain, only during the last decade (1920–1930), has undertaken to stipulate precautions to be taken in the construction and operation of dams, and to clarify the law with respect to liability for damage, in the event of a failure. The Italian regulations, as revised December 31, 1925, are perhaps the most comprehensive in force at present, in any country.

It is of interest that, in nearly every instance, the regulations were prompted by some dam catastrophe which impressed the necessity for such regulations upon the minds of the people.

Characteristics of Best Type of Supervision

Essential as the protection of life and property is, legislation to that end must be well considered along sound lines and must be neither difficult of compliance nor needlessly extravagant in its effect upon costs.

It is apparent that the failure of the St. Francis Dam caused governmental agencies concerned with the approval of dams to be considerably more exacting than they had been theretofore. In view of the apprehension engendered it is not improbable that the tendency will be to require dams to be constructed stronger than is actually necessary. Such excess strength can be had only from capital expenditures greater than have been required in the past, which in the case of hydro-electric power will impose even greater burdens than those which now obtain. Water powers are constantly growing more costly and one of the factors having much to do with this, is the growing State and Federal authority over water and water powers. It is conceivable that if this continues, water powers will finally become too costly to develop. Even now (1932), a water power must be most favorably situated to compete with the modern, highly efficient steam plant.

Engineers, public utilities, and other organizations interested in the building of dams, however, should take a rational, rather than an opposed, view on legislation, because there appears to be some justification for governmental control. Engineers, and particularly civil engineers, should determine for themselves the kind and extent of legislation desired. They should study the problem with a view to directing public opinion along lines not subversive of the best public interests. They should discourage anything in legislation which would tend to hamper the fine progress in the development of dam construction which has characterized the art.

As to the statute itself, it would seem desirable to foster the maximum use of existing governmental machinery—State engineers' offices for administration and the Courts for enforcement—rather than to create new agencies, except as providing for the services of consulting experts in major cases. Consulting boards acting during the design and construction periods only are not sufficient. Operation and maintenance must be checked by the routine inspections of some standing authority, after the consulting boards are discharged.

The statute, in so far as possible, should confine itself to the basic principles, leaving the details to be covered by regulations and rules which may be formulated by the State engineer or other administrator, as occasions develop. Few limitations should be placed upon the supervisory agency as to the tech-

nical aids which it may employ in the discharge of its functions. By these means necessary revision could be more readily effected than would be possible through legislative amendment that would otherwise be required. Furthermore, the statute should require promptness of action in the matter of approvals, either from State engineers' offices or other supervisory bodies which may be charged with its administration, in order that there may be no undue delay in gaining decisions on proposed construction. For instance, the Idaho law requires that approval be either given or refused within forty-five days.

The elements which should characterize the best type of supervision will fall naturally into certain groups which may be discussed more or less independently, bearing in mind that they all should have the common but limited objective of safeguarding life and property. Any governmental regulation which does not conduce to that end is needless and extraneous.

Extent of Supervision.—It seems that supervision must be had and it is probable that there is no way of getting it except through some governmental agency. If State supervision is to be established, it should be inclusive of the entire undertaking. It should involve, in so far as safety alone is concerned, the approval of the location, the design, the construction, and the maintenance of the dam. Such supervision should not involve the independent selection of the location or the design, the superintendency of construction, or the direction of maintenance on the part of the State, for the reason that such a procedure would result in a tremendous duplication of effort and expense and an unwarranted interference in the affairs and plans of the owners.

The supervision should be limited, therefore, to the approval of the location for the dam as selected by the builder, the checking of the design which he submits, and such occasional inspection as will insure the carrying out of the approved design and its maintenance thereafter. In keeping with this procedure, the builder of the dam should be required to submit a report of his investigation of the geological conditions of the dam site, a study of the flood run-off conditions of the water-shed behind the dam, as such study would affect the capacity of the spillway, together with plans and specifications of the dam proper, in sufficient detail to describe the nature and type of structure upon which approval is sought. He should be permitted to file supplemental plans and specifications with the same sufficiency of detail, to cover major changes in design which may seem desirable in the interests of safety, as subsequent information is revealed by the construction operation, or for any other good reason.

In addition to its responsibilities over new construction, the State authority should also have the power to condemn unsafe structures, to order water removed or lowered, or to limit permanently the height to which water may be stored, where existing dams are found to constitute a menace.

Exemptions from Supervision.—If the supervision of the design, construction, and maintenance of dams by the State Government is deemed essential in the public interest, there should be no exceptions because of ownership. Dams built by individuals, private corporations, quasi-municipalities, municipalities, counties, and the State itself, should be subject to the

same supervision. It is manifestly unreasonable to exempt the structures of municipalities and quasi-municipalities, on the score that they maintain organized engineering departments, and then to require the public utilities which also maintain engineering departments, to submit to supervision by State agencies. Dams built by the departments and bureaus of the Federal Government may not be subject to review by State authorities, due to lack of sovereignty.

Some exemption should be made for unimportant structures, the failure of which would not involve loss of life and property. This exemption, while governed largely by the volume of stored water, should also take into consideration the drainage area contributory to the reservoir and the height of the dam. The menace from bursting dams is largely a question of the volume of water released.

Supervision Personnel.—Those in supervisory capacity on behalf of the State Government must obviously be the peers in technical experience and judgment of those engineering, designing, and constructing the dam, in order that the decision of the supervisor may be unquestionably sound. State authority would accept no responsibility in case of a failure even if the structure had received its approval. Consequently, as authority and responsibility cannot be associated in such cases, authority must associate with itself unquestioned technique, experience, and judgment.

It is unfortunate that the compensation allowed engineering officers and consulting engineers in the service of the State imposes a hardship upon men of the best type when they forsake private practice for public appointments. It is even more difficult for engineers on standing consulting boards, who, in exchange for a small fee, find themselves precluded from practicing their specialty in the State which they serve.

It is patent that such State control should be vested in one office, possibly a bureau of the Department of Engineering. Not to centralize this responsibility would result in confusion as to authority, in lapses in jurisdiction over which no authority would exist, and in divided responsibility.

Burden of Cost

It is proper that the applicant for a permit to construct a dam should pay something into the State Treasury to help defray the expenses of the supervisory board; however, it would seem that the public at large should also pay something toward the security which it desires and receives, just as it does in connection with any other health and safety measure, within the police powers of the State. To this end, the cost of supervision and inspection of dams and the expenses of the bureau, board, or department that exercises the authority, should be divided as nearly as possible between the State on one hand and the owners and operators of dams on the other.

Some provision should be made respecting the burden of cost for inspections and investigations in the case of complaints alleging that the person or the property of the complainant is endangered by the construction, maintenance, or operation of a dam. While every consideration should be given to meritorious complaints, the State must safeguard itself against nuisance com-

plaints and must not permit itself to become an instrument of malicious persecution. To this end many States require deposits of complainants, returnable in the event that the complaint is well founded.

Appeal Provision

There are bound to be cases where the builder of a dam would be reluctant, in the interest of safety or economy, to follow the decision of the supervisory control. It is conceivable that an individual, corporation, or municipality may actually regard the dam required to be built by the supervisory control as a less safe structure, or one for which the expenditure would be more than need be, to safeguard the public, compared with that which they propose and for which a permit might have been denied. Therefore, in order to avoid situations which would leave no alternative to those contemplating the building of dams, but to build that dam which would be approved by the State Supervisor, provision should be made in any legislation on the subject for an appeal to the Courts from the decision of the State agency, by any party in interest who may be dissatisfied with any decision. The issues on appeal should be heard at the earliest practicable time and determined summarily without a jury. The decision of a Lower Court should be subject to appeal or review as in other cases.

In addition to the right of appeal to the Courts, an alternative method of review could well be provided, namely, the right to present the questions at issue to a board of engineers acting as arbitrators, one of such board to be selected by the State Engineer, one by the builders of the dam, and a third by the first two named. The decision of this board should be final. The expense of the hearing before this board should be divided in every case between the State and the party making the reference, since the State would never make the reference in its own behalf, and, otherwise, the cost of the arbitration would fall, invariably, upon the private appellant.

Existing Dams

The fact that a given structure has been standing for a period of years and during that time has successfully met unusually severe conditions, generally should be taken as conclusive evidence of satisfactory construction. However, investigation of the operating history of the many existing dams as to their performance during their lives may be desirable. Should investigation disclose structures that have been on the border line during periods of stress, or if there are found those which should be replaced because of depreciation, the owners might be called upon to show cause why such structures should not be replaced. Consequently, any legislation that may be enacted in any State looking to the supervision of dams should be broad enough to cover the inspection of existing dams and confer power on the supervisory agency to compel owners of such dams as are not now safe, or which may become unsafe, to repair or reconstruct them to any reasonable extent that may be found necessary to insure their safety.

It is probable that it would be unnecessary to inspect all the many existing dams, some of which are small and impound only small quantities of water.

Every owner of a dam, however, could well be called upon to furnish a description of it, of all works connected with it, an estimate of the water impounded, and a statement of the tributary drainage area. After such reports are received, those particular dams—the failure of which would be likely to endanger life or property—should be inspected, and if any are found unsafe the owners should be compelled to repair or reconstruct them as may be necessary. Plans for the reconstruction or modification of existing dams would be subject to the same approvals as are required for new structures.

Maintenance of Dams

While a structure, when built, may be of satisfactory design and construction, its continuance in a good state of preservation is, of course, another matter, and while a well-constructed dam should remain in safe condition under intelligent operation and with reasonable maintenance, it would doubtless be advisable to establish a routine inspection, perhaps biennial, over all important structures. Such an inspection would operate to guard against malicious and insincere complaints respecting existing structures.

The very massiveness of earth, rock-fill, and masonry dams prompts the belief that they are of everlasting life. Lacking definite data respecting the erosive and destructive action of the elements on dam materials over long periods of time, the safe working life of such construction cannot be established as yet. It is possible that serious deterioration, more or less unsuspected, has been taking place in some of these structures built several decades ago. State supervision, due to its perpetuity, offers a continuing control over structures of this type, outlasting the lives of the designers and builders.

Building Code for Dams

In the foregoing no mention has been made of a code to cover the building of dams, because it is probable that legislation will revolve around the idea of supervisory personnel rather than rules of good construction. However, in other types of structure, experience has dictated the wisdom of building codes. While it is recognized that there are many disadvantages in the codification of anything, and that a code for the building of dams would be more difficult to devise than a code for buildings, it is not impossible, should a code be considered otherwise desirable, that a competent group of engineers could fashion a comprehensive and workable one, at least as far as certain important features for the building of dams are concerned. Such a code could be administered by the department of engineering of a State desiring one. This idea suggests itself because of the possibility of outlining the conditions under which the generally accepted types of dams would be best suited. The well-known types of dams include: (*a*) Earth dam (rolled or hydraulic-fill); (*b*) rock-fill (dropped rock or rubble rock); (*c*) rock masonry; (*d*) concrete (gravity, arch, multiple-arch, or buttressed slab); (*e*) crib (generally wood and occasionally concrete); and (*f*) frame (steel, wood, or concrete). All these types have their peculiar adaptability, taking into consideration location (topography, geology, and altitude), height, availability of material, desired life, foundations, disposal of floods, and other factors.

It would be possible to prepare a code to cover the construction of dams, which would describe the conditions under which each of the types could be built, from which a builder could select one, providing the economics in the particular situation did not make the structure entirely unfeasible. If it did, nothing could be built. Codification probably would not dispose of differences of opinion, in individual cases, as to the safety of the structure which might be required by a board administering such a code. An arbitration board would be valuable here also, but it is probable that the action of such a board would be infrequently required. The code could be in the nature of a minimum requirement, which responsible builders would tend to exceed in their own interests, having in mind the protection of their investment.

This suggestion of codification is offered for what it is worth. It might prove, after consideration, that the conditions to be covered would be too diverse to permit of practicable definition in such a code.

Conclusion

Any one who visited the site of the St. Francis Dam after its failure could not but be profoundly impressed by the death-dealing effect of the released flood. Those who were in the path of this flood, unconscious martyrs as they were, gave their lives for the benefit of mankind. Nothing great in the world has ever been accomplished without sacrifice; it is to be hoped that the hundreds in this case have not given their lives in vain.

The loss of life through the failure of any dam is deplorable. It is not more deplorable than the loss of life which occurs through modern instruments of production and transportation. No one would consider for a moment abolishing automobiles because of the tremendous loss of life which takes place through their existence. Consequently, a disaster like that of the St. Francis Dam must not occasion unnecessary capital investments and of such an order to prohibit the conservation of water for the various uses to which it is put.

The fears which have been raised throughout the United States must not be allowed to prohibit the construction of dams. Engineers must not be stampeded because of unfortunate happenings, because they know that very high dams to impound large volumes of water can be built safely, provided proper engineering principles are used. Technique of engineering design need not cause concern because of the abundance of high-class technical knowledge and skill that is available. Common sense and judgment, however, are rare qualities, the one being inherent and the other to be had only through experience.

The general subject of dams and the kind of legislation relating to supervision and control which will safeguard the public on the one hand, and not impose conditions which would cause unnecessary delay, excessive cost, or prohibition on the other, would be a valuable subject for further study by engineers. It would perhaps be timely for such a study to include an outline of a type of uniform legislation which it would be best for the States to enact, since the conditions which it is desirable to control in the various States are not so diverse as to preclude the successful operation of such a uniform act.

NECESSITY FOR, AND PENALTIES FOR LACK OF, SUPERVISION

By M. C. Hinderlider,[4] M. Am. Soc. C. E.

Synopsis

It is not the purpose of the writer to go into the question of the economics of dam construction, nor to discuss the many intricate, albeit fascinating, questions involved in theories of dam design, but rather to endeavor to discuss the subject more from the viewpoint of the public which, after all, is the party most vitally interested in the design, construction, maintenance, and operation of dams. This superior interest arises from two causes: First, that having to do with the element of hazard; and, second, that having to do with the economic advantages to be secured through the construction and operation of storage reservoirs.

The paper contains a tabulated summary of 293 dams that have wholly or partly failed, and a classification of the causes of these failures; it also contains a general comment on existing laws in the various States, and a suggestion for framing a code for the design and construction of dams.

Fifteen items are included in the paper to indicate subjects that should, or should not, be included in State supervision; twelve items portraying the advantages and disadvantages of State control of dams, and twelve items defining the ideal qualifications of supervising officials.

Basic Need for Supervision

Every dam, regardless of its size, is in some degree a potential menace to everything below it. There is nothing so relentless in its immediate destructiveness, so uncontrollable and deadly as a huge volume of water suddenly released. The effects of cyclones, earthquakes, and even volcanoes are generally local, and their occurrence is infrequent, and such menaces usually furnish warning of their approach. Great conflagrations are subject to control by modern methods, and science has developed means for successfully combatting the ravages of diseases and epidemics; but no means will ever be contrived for overcoming the ruthless destructiveness of huge bodies of water suddenly released from restraint.

The number of recorded failures of dams, however, will doubtless compare favorably with the failures of other engineering structures similar in magnitude and in menace to life and property. Nevertheless, such failures have

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been all too frequent, and constitute a reflection upon the ability of the Engineering Fraternity. Fortunately, or unfortunately, the psychological result of such failures is of short duration; the effects are soon forgotten until the tragedy is re-enacted.

Table 1 is a summary of reported partial or complete failures, since 1799, of 293 dams located throughout the United States and in some foreign countries. Such failures represent various types ranging in height from 10 to 190 ft. The list includes 159 earth dams—9 of which were of the hydraulic-fill type; 12, rock-fill; 67, gravity masonry type; 7, single and multiple-arch; 7, reinforced concrete; 1, steel; and 7, timber; and 23 other dams of unclassified types. It includes failures of spillway, abutment, cut-off, or any other appurtenant part of a dam that may have jeopardized life or property. A critical study of these data should be made by every engineer who assumes the responsibility of designing or supervising the construction of a dam, no matter of what type or magnitude. Table 1 discloses that inadequate spillway facilities and defects in outlet works were the causes of most failures of earth dams, while foundation troubles were the principal causes contributing to the failure of masonry dams, or earth dams with masonry cores.

A tabulation and bibliography of the failure or partial failure of 293 dams of different types have been placed in the Engineering Societies Library, 33 West 39th Street, New York, N. Y. Photostatic copies may be obtained at cost.

The State of Pennsylvania heads the list with thirty-three recorded failures; twenty-six failures of dams are reported from the State of California; twenty-four from Colorado; and twenty-five from New York State. Eleven failures are reported from Connecticut, thirteen from Massachusetts; twelve from Missouri; seven from Ohio; six from each of the States of Michigan, Minnesota, and Texas; five from each of the States of New Mexico, North Carolina, Arizona, and Utah; four from each of the States of Montana, New Hampshire, New Jersey, Rhode Island, South Carolina, and Maine; three from each of the States of Georgia, Oregon, and Tennessee; and a total of thirty from all other States. The record also includes forty dam failures in foreign countries.

Doubtless this tabulation does not include all total or partial failures of dams in such States and countries, and in the absence of a complete record showing the relative percentages of failures, significance attaching to State supervision is lacking. However, in so far as the analysis is partly complete, it discloses that the largest number of failures have been in Pennsylvania, California, Colorado, New York, Wisconsin, Massachusetts, and Connecticut, all of which States have exercised supervision over the construction and maintenance of dams for many years. The largest number of failures, however, might be expected from those States which have, by far, the largest number of dams in operation. In California, divided or complete lack of State authority probably has had something to do with so many failures. Colorado, which has more than 1 000 dams under the supervision of the State Engineer, has been a pioneer in such construction, which has provided

TABLE 1.—LIST OF TOTAL, PARTIAL AND INCIPIENT DAM FAILURES, 1799–1931, INCLUSIVE.
(Compiled December, 1931.)

Reference No.	Cause of Dam Failures.	Reference No.	Cause of Dam Failures.
1	Inadequate spillway.	8	Ice pressure or disintegrating effect of.
1(*a*)	Inadequate spillway. Overtopping by flood wave due to failure of dam above.	9	Improper operation or inadequate maintenance.
2	Inadequate cut-offs. Porous foundation allowing leakage and erosion under earth dam, and/or sliding in rigid types.	10	Burrowing rodents.
3	Faulty construction; material not properly compacted in earth dams; poor construction in masonry dams.	11	Poor materials, including soluble salts.
4	Inadequate cut-offs around conduits in earth and rock-fill dams.	12	Unstable or structurally weak foundation.
5	Faulty design of section; slopes too steep in earth dams; section too light in masonry dams.	13	Conduits through earth or rock-fill dams not properly supported to prevent settlement or failure. Improper location of valves.
6	Inadequate means for stream control during construction.	14	Insufficient provision against erosion from back-wash below dam or spillway.
7	Excessive quantities of clay or other classes of fine material.	15	Earthquakes.
		16	Miscellaneous and undetermined.
		17	Failure of bottom in small water-works reservoirs.

SUMMATION OF DAM FAILURES FROM DETAILED LIST WHICH FOLLOWS. NUMBERS IN PARENTHESES DENOTE PERCENTAGES OF THE TOTAL IN EACH CASE.

Type.	Height, in feet.	Number of Failures Under Each Cause. Reference Number of Cause Refers to Above List.																		Total, percentage.	
		1.	1(*a*).	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.	15.	16.	17.		
Earth	0– 25	15	4	4	1	5	....	....	....	....	....	....	....	....	....	1	....	4	....	34	(21)
	25– 50	16	1	5	....	5	2	2	2	....	1	....	....	....	....	1	1	1	....	37	(23)
	50– 75	3	....	5	4	2	3	2	2	....	....	1	....	....	....	1	....	3	....	26	(16)
	75–100	....	....	....	....	1	....	....	2	....	....	....	....	....	....	....	1	2	....	6	(4)
	100–125	....	....	....	1	....	1	1	1	....	....	....	1	....	....	....	....	1	....	6	(4)
	125–150	....	....	....	....	....	....	....	1	....	....	....	....	1	1	....	....	....	....	3	(2)
	150+	....	....	....	1	....	....	....	2	....	....	....	....	....	....	....	....	2	....	5	(3)
	Not given	10	3	4	1	5	....	....	2	....	1	1	2	....	1	....	1	5	6	42	(27)
Total		44	8	18	8	18	6	5	12	0	2	2	3	1	2	3	3	18	6	159	....
Percentage		(28)	(5)	(11)	(5)	(11)	(4)	(3)	(8)	(0)	(1)	(1)	(2)	(1)	(1)	(2)	(2)	(11)	(4)	(100)	

TABLE 1.—List of Total, Partial, and Incipient Dam Failures.—(*Continued.*)

Type.	Height, in feet.	Number of Failures Under Each Cause. Reference Number of Cause Refers to Above List.																		Total, percentage.	
		1.	1(*a*).	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.	15.	16.	17.		
Rock-fill	0– 25	.....	.....	1	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	.....	3	(25)
	25– 50	2	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	3	(25)
	50– 75	.....	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	(8)
	75–100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	0	(0)
	100–125	1	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2	(17)
	125–150	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	(8)
	150+	.....	.....	.....	.....	.....	.....	1	.....	.....	.....	.....	.....	.....	1	.....	.....	.....	.....	2	(17)
Total		4	0	1	2	0	1	1	0	0	0	1	0	0	1	0	0	1	0	12	.....
Percentage		(33)	(0)	(8½)	(17)	(0)	(8)	(8½)	(0)	(0)	(0)	(8)	(0)	(0)	(8½)	(0)	(0)	(8½)	(0)	(100)	.....
Masonry gravity type	0– 25	.....	.....	7	2	.....	3	.....	.....	5	1	.....	.....	.....	.....	.....	.....	3	.....	21	(31)
	25– 50	2	1	3	5	.....	4	.....	.....	1	1	1	.....	.....	.....	.....	.....	2	.....	20	(30)
	50– 75	.....	.....	2	1	.....	.....	.....	.....	.....	.....	.....	.....	1	.....	.....	.....	2	.....	6	(9)
	75–100	1	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2	(3)
	100–125	2	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2	(3)
	125–150	.....	.....	.....	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	.....	2	(3)
	150–200	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	(1½)
	200+	.....	.....	.....	.....	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	(1½)
	Not given	1	.....	7	.....	.....	.....	1	.....	.....	.....	.....	.....	1	.....	1	.....	1	.....	12	(18)
Total		6	1	21	8	0	8	2	0	6	2	1	0	2	0	1	0	9	0	67	.....
Percentage		(9)	(1½)	(31)	(12)	(0)	(12)	(3)	(0)	(9)	(3)	(1½)	(0)	(3)	(0)	(1½)	(0)	(13½)	(0)	(100)	.....

TABLE 1.—List of Total, Partial, and Incipient Dam Failures.—(*Continued.*)

Type.	Height, in feet.	Number of Failures Under Each Cause. Reference Number of Cause Refers to Above List. 1.	1(a).	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.	15.	16.	17.	Total, percentage.	
Arch — single and multiple..........	0– 50	....	....	....	....	....	....	....	....	1	....	....	....	....	....	....	....	....	....	1	(14)
	50–100	....	....	....	1	....	....	....	....	1	....	....	....	2	....	....	....	....	....	4	(57)
	100–150	....	....	....	1	....	....	....	....	....	....	....	....	....	....	....	....	....	....	1	(14½)
	150+	....	....	....	....	....	1	....	....	....	....	....	....	....	....	....	....	....	....	1	(14½)
Total..........		0	0	0	2	0	1	0	0	2	0	0	0	2	0	0	0	0	0	7	......
Percentage..........		(0)	(0)	(0)	(28½)	(0)	(14½)	(0)	(0)	(28½)	(0)	(0)	(0)	(28½)	(0)	(0)	(0)	(0)	(0)	(100)	......
Reinforced concrete....	15– 51	.... (0)	.... (0)	6 (86)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	1 (14)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	7 (100)	......
Steel..........	70	.... (0)	.... (0)	1 (100)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	1 (100)	......
Timber..........	1 dam ,100 16 dams, less than 100	2 (12)	.... (0)	4 (23)	5 (29)	.... (0)	.... (0)	1 (6)	.... (0)	.... (0)	2 (12)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	.... (0)	3 (18)	.... (0)	17 (100)	......
Not given..........	Not given.	2 (9)	.... (0)	1 (4)	1 (4)	.... (0)	1 (5)	.... (0)	.... (0)	1 (4)	.... (0)	.... (0)	.... (0)	1 (4)	.... (0)	.... (0)	.... (0)	16 (70)	.... (0)	23 (100)	......
Grand total..........		58	9	52	26	18	17	9	12	9	7	4	3	6	3	4	3	47	6	293	......
Percentage..........		(20)	(3)	(18)	(9)	(6)	(6)	(3)	(4)	(3)	(3)	(1)	(1)	(2)	(1)	(1)	(1)	(16)	(2)	(100)	......

ample opportunity for mistakes. However, with the exception of the failure of two small earth dams, each 17 ft. in height, no failures have occurred in the State of Colorado in the eight years since 1923.

Every dam is in some degree a menace to life and property, yet these structures are indispensable to the welfare of the people. As such, their safety is a matter of such concern to the general public as to bring their supervision properly within the scope of the police prerogatives inherent in all civilized nations.

Legislative Trends

A digest of the laws of all the States, and the principal countries of the Old World, pertaining to dam design, as might be expected, strongly indicates the close relation between the economic importance attaching to the regulation of water supplies of such countries, and the degree of public supervision exercised over the design and construction of dams therein.

This police power or authority is defined[5] as "the power vested in the legislatures to make such laws as they shall judge to be for the good of the Commonwealth and its subjects. The power to govern men and things, extending to the protection of lives, limbs, health, comfort, and quiet of all persons and the protection of all property within the State." The exercise of this power of a State or Nation to protect the health and lives of its citizens from all kinds of evils and dangers has spread rapidly during the last fifty years.

Such police powers are very broad, as they must necessarily be in order that the objective sought shall be attained. Such protection is thrown around the citizens of practically all civilized countries and extends to factories, mines, packing plants, dispensaries, transportation systems, buildings and structures for whatever purpose used, and within comparatively recent times, to lines of agricultural development, food supplies, and for the control and eradication of contagious and infectious diseases, and, in fact, wherever the safety and well-being of the citizen and his property is beyond his individual ability to insure. Such supervision on the part of a State is the gradual outgrowth resulting from increase in population and the complexities incident to the development of civilizations the world over. It is a function of Government which is becoming increasingly great and of apparent necessity in the evolution of the social, economic, and commercial development of a nation. Its beneficial aspects are readily recognized, although having the effect of subordinating individual liberty in the greater interest of public welfare.

Public supervision over the building of dams is exercised in some degree in most of the States. The degree of such supervision is found to be generally in the proportion that regulation of stream flow has progressed in any particular State. The centralization or co-ordination of such public control varies widely in many of the States and in foreign countries as well. Centralized control is being more and more recognized as a public necessity. Such tendency is evidenced in the States of Pennsylvania, New York, Cali-

[5] Bouvier's Law Dictionary.

fornia, and Colorado, which report the largest number of dams. Until 1928, public supervision of dams in California was vested in four State and Federal Departments; in addition, municipalities under certain conditions were exempt from any public supervision whatever.

For many years the supervision of the design, construction, maintenance, and operation of all dams in Colorado more than 10 ft. in height, has been vested exclusively in the State Engineer, whose decisions in such matters are practically final.

In matters of such vital concern as the safety of a dam, even of nominal height, theories and ideas of a single mind should not be considered conclusive. Regardless of the extent of the study and experience of the individual engineer or geologist, each has had different problems to meet and overcome in traveling the rough road of a professional career. It is the well-co-ordinated results of such schooling that contributes in the greatest degree to the safety of the public. The conditions surrounding the design, location, and construction of every dam, has its own peculiarities which should be studied, analyzed, and weighed from all points of view and under all the probable conditions under which the structure is likely to function. It is rarely possible to find in a single individual a combination of experience involving full knowledge of design, construction methods, and materials, geology, hydrography, effects of temperature changes, disintegration, and the many other elements which contribute to the safety or weakness of a dam. The writer is convinced that the average practicing civil or hydraulic engineer is generally unfamiliar with many of the fundamental requirements essential to the rational design, supervision of construction, and maintenance of such structures, and that the usual "factor of safety" in design customarily adopted is indeed a saving factor to those whose lives or property may be placed in jeopardy by such structures.

In the interest of public welfare, many States have legalized the practice of engineering in its various branches by statutory provision, but, unfortunately, such provisions fall short of affording the degree of protection contemplated. Contrary to a reasonable assumption on the part of the general public, the reverse may even be true. With rare exceptions has the writer felt justified in approving either the plans or specifications for dams built in Colorado until they had been revised in whole or in part, and it is presumed that a similar condition may exist in other States. Just how far the authority of the approving official should extend in indicating the type of dam and appurtenant structures, character of construction, etc., is a mooted question.

In Colorado, the State Engineer follows the practice of not only suggesting the type of structure indicated by the site and geological formation, run-off conditions, availability of construction materials, etc., but for the purpose of conserving the time of, and expense to, the owner, he recommends to the engineer that a preliminary draft of his plans and specifications be submitted for criticism prior to the preparation of the final draft for approval. This plan has seemed to work well in practice. Study and practice relating to

the design of practically all conservative types of dams have resulted in the development of principles generally recognized as being sound, although there is still some diversity of opinion with respect to important theories affecting the safety of huge structures of the gravity and lighter-than-gravity types of masonry dams. This is to be expected since much remains to be learned concerning the physical and chemical properties of building materials, their behavior under combinations of loading, temperature changes, chemical reactions, and variations in construction details, all of which are impossible of complete control.

As in the evolution of the development of all the arts and sciences, the proof of a theory is largely dependent upon its successful application in practice. In the earlier history of dam construction, such structures were usually built of materials to which scientific principles, even, if they had been understood, were impossible of application. In general, the evolution of scientific knowledge in the field of engineering has not provided a greater degree of safety in the construction of dams of modern height, but it has vastly increased the range, not only in the magnitude of such structures, but in the ability of the engineer to overcome the deficiencies of Nature by the adaptation of proper types of structure, to any particular condition.

Doubtless, the greatest value attaching to present-day knowledge of the principles of dam design and construction, is one of economics. Such knowledge has made it possible to utilize storage sites formerly considered infeasible, and through such use of basins of great capacity, stream regulation upon a large scale has been made feasible. Under the urge for development of natural resources the world over, structures of ever-increasing magnitude are coming into existence and are no sooner completed than even larger ones are conceived and planned for the future. The engineer is called upon to meet this urge in a way that bids fair to tax to the limit his ingenuity, good judgment, and conservatism. He must of necessity be something of a dreamer, or he would be unable to originate new ideas and evolve the great undertakings demanded by modern civilization. He is inherently a conceiver and builder. He is generally an optimist, at times prone to allow himself to be unduly influenced by a desire to achieve great undertakings, sometimes under conditions in which safety to life and property is dependent upon his good judgment. Probably in no field of endeavor are the responsibilities to the public so great or exacting as in that of the design, construction, and supervision of structures for the impounding of large bodies of water. It is well, therefore, that those charged with such obligations should proceed with caution, be guided by a high degree of conservatism, and withal imbued with an exalted sense of their responsibility. The public is prone to defer to the judgment of those, presumably competent, to look after its interests, and to go about its own business of fighting the battles or seeking the pleasures of life. It is lulled into a sense of security through its aversion to being bothered, until some great catastrophe is precipitated upon it, and then it usually acts under the stimulus of stress and hysteria.

The responsibility resting upon a public official in charge of dams is indeed a heavy one, not only as a result of his approval of the plans for such structures, but because of possible internal weaknesses in dams with which he may have had no former connection. Fortunately, as in the case of human diseases, certain symptoms usually warn the careful observer of trouble ahead, and therein lies one of the best arguments for centralized responsible public supervision over such structures, which enables some one to act promptly and with authority.

A review of the statutes of all the forty-eight States, including the District of Columbia, discloses that, with the exception of Delaware, Louisiana, Minnesota, Nebraska, Oklahoma, and the District of Columbia, they all exercise some form of public supervision over dams, ranging from that relating to damages which may result from the action of back-water above a dam, and the imposing of regulations in the interest of fish propagation, navigation, and sanitation, to the higher purposes of protection to life, property, and industries which may be placed in jeopardy. Supervision is vested in various agencies, ranging from Courts, municipalities, commissions, and counties, to the State, and departments of the National Government. As is to be expected, the degree of such regulation increases with the need for dams in the economic and commercial development of the State or country. In many of the States where the topography of the country and gradient of the streams are conducive to power development in only a minor way, the need for public regulation is not so urgent, while in those States and countries in which conditions and demands are favorable to the greatest utilization of their water resources, will generally be found the greatest degree of public supervision.

The aforementioned review of the laws of the forty-eight States disclosed that, in nine States, supervision of dams is vested in the Courts; in six States, in the Board of County Commissioners or Supervisors; in fourteen States, in State Commissions or Boards; and in fourteen States, it is placed in the hands of a State Engineer or his equivalent. In several of the Public Lands States, some Government agency, such as the Bureau of Reclamation, Federal Power Commission, Débris Commission, Indian Service, or Forestry Department, exercises partial or joint supervision over the design, and, at times, over the administration of dams. The Army Engineers exercise supervision over all dams where navigation is affected, or in the interest of the protection of post routes.

Public supervision extends to structures for practically all purposes, and of all heights, ranging from 5 ft. upward. Supervision relates in many States not so much to the safety features of such structures as it does to matters of navigation, power development, stream pollution, and flood control. From the standpoint of safety, supervision is usually exercised by the State in which the structure is located, although, in several States, dams built and operated by the Federal Government are exempt from State supervision.

Analysis of the laws of all the States discloses that inspection and supervision over dams in the interest of public safety is exercised only in nineteen

of them. Of these, fourteen have regulatory laws which may be considered reasonably effective. In only twelve of the States may existing laws relating to State supervision be considered ample for the safguarding of the public's interest where dams of considerable magnitude are contemplated. In more than one-half of all the States there are either no laws relating to public control of dams which may be considered a menace to life and property, or such laws as exist are for the minor purposes heretofore mentioned. Accordingly, it would appear desirable that in such States careful study of this subject of public supervision of dams should be made. Practically all States of the Rocky Mountain region and the Pacific Coast, have quite effective laws relating to this most important matter. This condition also applies to several of the Atlantic Seaboard States, such as New York, New Jersey, Pennsylvania, Maine, New Hampshire, Vermont, Connecticut, and Rhode Island.

The Engineer's Responsibility as Regards Legislation

Legislation affecting the construction of dams invites the close attention of engineers. It is of far-reaching importance and should not be passed preliminary to careful and mature consideration of the objectives to be attained and the difficulties to be overcome. The engineer, who is trained in the theory of stresses and strains and the characteristics and frailties of the materials with which he must work, is fully appreciative of the real magnitude of such undertakings and of the tremendous responsibilities they represent. Activity on the part of engineers, who by study and experience are justified to speak on this important subject, is a public service which the engineer could, and should, render. No one is so qualified as he, to point out the dangers and to indicate the way.

Some of the disadvantages resulting from public supervision over the design and construction of dams are attributable to the fact that much of such legislation was enacted without adequate information, or that it is the result of sporadic attempts to correct, from time to time, deficiencies in previous legislation. This has frequently resulted in contradictions, ambiguities, or complete lack of some essential, and too often has resulted in the objectionable features of divided authority. Consequently, the laws of many States relating to this most important subject are a "hodge podge," making for inefficiency in their administration, with the resultant unwarranted sense of security which the public has every right to expect under State supervision. The success of any law depends largely upon the degree of public approval back of it, and upon the confidence reposed in the official charged with the duty of administration.

It is also apparent that the beneficial effects of any law are measured by the degree of efficiency with which such law may be executed or administered and, regardless of the safeguard which a law may seek to establish, the effectiveness will depend very largely upon its enforcement. It is just as apparent also that the actions of a public official are circumscribed by the limitations and weaknesses of the law which he is charged with enforcing. The first requisite, therefore, are laws as nearly ideal as may be, for the

attainment of the desired objective. Hence, the desirability that laws relating to the design and construction of dams should reflect the best engineering thought and experience.

Codes for Design and Construction of Dams

A comprehensive code covering the design and construction of dams may well be included in such studies. It is realized that constitutional provisions in many States may render the application of a general code difficult, if not impossible, but doubtless it would be invaluable as a background for the framing of needful legislation applicable to conditions in some particular State or country.

It is believed that fundamental principles relating to stresses, strains, reactions, and the behavior of materials used in the construction of masonry dams of various types, and modern methods relating to construction, are sufficiently well understood and demonstrated, to justify the formulation of a code of procedure which would receive the approval of the great majority of engineers familiar with this subject. A uniformity of practice based upon such a code would go a long way toward minimizing the number of failures of dams, which would certainly justify the endorsement of the profession. No type of structure involves the degree of hazard to the public as that of storage dams. The public is entitled to every consideration in this regard, but can obtain the necessary degree of protection only through proper laws, which should embody the consensus of the best thought and experience available. Within recent years, and as a result of intensive studies by the constituent bodies of Engineering Foundation and other similar organizations, uniformity of practice and procedure, with respect to building materials, many types of structures, and related matters, has been adopted and recognized in almost all lines of engineering endeavor. A code of requirements relating to the art of dam building would be invaluable in the drafting of legislation providing for public supervision over the design and construction of dams.

A code[6] proposed by Fred A. Noetzli, M. Am. Soc. C. E., provides an excellent foundation upon which to predicate a practical and effective set of rules which, with certain additions or modifications, could be embodied in future legislation on the subject of dam construction.

Of scarcely less importance in the public interest, is the matter of inspection and maintenance of dams. The evil effects of faulty design and construction may generally be met successfully by constant and careful inspection, operation, and maintenance, even under trying conditions. The authority of the public official in this regard covers many acts which must necessarily be of a discretionary nature. Such acts directly affect the property rights of the owner and, at all times, the degree of safety to lives and property below a dam. Just what depth of water it may be safe to store back of a dam under certain given conditions, or how long such storage should be permitted in the case of earth dams, or what margin of free-

[6] *Western Construction News*, September, 1929.

board should be required on many dams (particularly during periods of high wind velocities peculiar to the arid regions), the proper size of spillway to maintain, and many similar items are problems which continually arise to perplex the administrative official. He is charged with the duty of safeguarding life and property below a dam, and, at the same time, he must recognize and permit the maximum utilization of the investment represented by such structure.

Since the foregoing problems are quite complex in their nature and are continually in a state of flux, it does not appear practicable to formulate a code of rules or laws applicable to the maintenance or operation of dams except along general lines. The effectiveness of administration very largely depends upon the authority, experience, and sense of duty of the public official, and, therefore, cannot be fixed by any given set of regulations.

The Act[7] passed by the Legislature of California in 1929, providing for State supervision over all dams other than those built by the United States Government, is an excellent piece of legislation on this subject. The California law is the result of much study and discussion and, with minor exceptions, will doubtless be found to be highly effective and practical in its application. This law is largely the result of the crystallization of public opinion occasioned by the St. Francis Dam disaster and reflects quite generally the most modern thought on the subject of State supervision of dams.

Qualifications of Supervising Officials

The qualifications of the official charged with the important duties of State supervision of dams should include:

(*A*) An understanding of the fundamental principles affecting the stability of all types of dams. (This requirement, in general, pre-supposes that he is a technical graduate or, because of previous preparation, is competent to understand the mathematics of engineering design.)

(*B*) An appreciation of the advantages of the science of geology and its adaptation to the art of dam building. (He should also be thoroughly grounded in the knowledge of climatology and hydrography.)

(*C*) Broad experience in the problems of construction (which is conducive to a discriminative sense of values and dangers), and of materials, their properties, and uses.

(*D*) A willingness to improve his knowledge of the art by consulting with and studying the methods and theories of others. (He should be imbued with an honest desire to learn the lessons taught by the failures of the past.)

(*E*) An exalted sense of his duty to the public and the ability to visualize the grave responsibilities with which he is charged.

(*F*) Complete freedom from any influences other than the motivating ones of his official duties.

(*G*) Tenacity of purpose.

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[7] Chapter 766, Laws of 1928–29.

(*H*) An appreciation of the value of systematic methods in the conduct of his office and the essential need for reliable data at all times.

(*I*) Progressive ideas, albeit leavened with that sense of responsibility which his duties impose.

(*J*) The confidence of his fellow engineers and the public.

(*K*) A genuine desire to co-operate, but the stability of purpose not to allow his enthusiasm to get the better of his sober judgment; and, finally,

(*L*) That rare combination of natural ability and experience commonly known as "horse sense."

Itemized Conclusions

The following suggestions are offered by the writer with the hope that they may elicit further discussion of this most important subject of public supervision of the design and construction of dams.

A law for State supervision of dams should include authority to:

1.—Pass on the design, construction, maintenance, and operation of all dams and reservoirs of a certain minimum height and capacity.

2.—Require (preliminary to beginning of construction), complete data on the geology, topography, rainfall and run-off characteristics of the drainage basin, samples of materials from foundations and those to be used in construction, and all other information needful for a proper determination of the suitability of the dam site.

3.—Require complete plans, specifications, and analyses of design, and also authority to modify such plans and specifications prior to and during the progress of construction when conditions would seem to justify such changes.

4.—Require continuous inspection of new construction and all materials entering into it, preferably at the expense of the State.

5.—Employ consulting engineers and geologists either upon the authority of the State, or upon the request of the owner, such authority to apply not only to contemplated construction and dams under construction, but also to all existing dams.

6.—Exercise supervisory control during construction and operation of the dam following construction.

7.—Regulate at all times the quantity of water stored back of any dam.

8.—Require all dams to be maintained in a safe condition.

9.—Enforce all provisions of the law with respect to the construction, operation, and maintenance of dams, with recourse to the Courts only under exceptional conditions which would require a prompt determination of the matter by the Court.

10.—Provision for ample funds to carry out State supervision as herein noted.

State supervision should not include:

I.—Financial obligation on the part of the State in case of failures.

II.—Imposition of unnecessarily drastic regulations not essential for safety.

III.—Unnecessary interference with methods of construction.

IV.—Authority to assume the functions of the practicing engineer with respect to the preparation of plans and specifications and related data required in connection with the design and construction of a dam.

V.—Authority for State supervision in too minute detail. In regard to Item V, State supervision should be limited to the approval or disapproval of the plans and data presented, and the authority to require essential amendments thereto, which, in the judgment of the State official, appear to be needful for insuring the proper degree of safety, and to provide for adequate inspection during the period of construction.

State control of the design, construction, and maintenance of dams has the following advantages:

(*a*) Centralized authority with its singleness of purpose and responsibility to the public.

(*b*) Minimum of effects of local and political influences.

(*c*) Prompt and concerted action in anticipation of dangers and the forestalling of failures under most conditions.

(*d*) Uniformity of procedure in design and inspection.

(*e*) Greater assurance of disinterested inspection.

(*f*) Co-ordinated control in administration of the uses of stored water and stream flow, especially in arid regions.

(*g*) Continuity and permanency of records and public accessibility to them.

It has the following disadvantages:

(*h*) Danger of incompetence of, or prejudice on part of, State official.

(*i*) Lessening of the sense of responsibility and precaution on part of owner.

(*j*) Possibility of injustices to owner resulting from unwarranted requirements of State official.

(*k*) Erroneous assumption on the part of property owners below a dam that State supervision is tantamount to State responsibility in a pecuniary way.

(*l*) Dangers accompanying increase in bureaucratic authority.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

STEREO-TOPOGRAPHIC MAPPING [1]

By C. H. Birdseye,[2] M. Am. Soc. C. E.

Synopsis

This paper begins with a general introductory statement of the relative advantages of aerial and ground-survey methods in topographic mapping. A discussion of the fundamental principles of stereoscopic measurements is given to prepare the reader for a better understanding of the mechanical features involved in the construction of measuring stereoscopes. A description is given of some of the instruments in use at present for making topographic maps from photographs taken in the air. The theory of the aerocartograph is described in detail because that is the instrument with which the writer is most familiar. However, it is intended that this description be considered merely the "ground work" for a general theory and that discussion will produce comments on similar instruments in so far as they vary from the aerocartograph, on the advantages and disadvantages of the various types, and on general suggestions for improvement based on experience.

Introduction

It is recognized generally that the surface of the ground can be depicted by a proper use of aerial photography with a greater degree of faithfulness, especially as regards minor detail, than can be accomplished by the exclusive use of ground-survey methods. Modern aerial-survey methods will provide the essential information required on many engineering works, such as railroad, highway, and transmission-line location, and will eliminate many trial ground surveys at a saving of both time and money.

These modern methods are particularly well adapted to aerial mapping of regions in which the drainage, culture, or relief is complex and full of minor detail. Too often the engineer has seen the results of surveys by transit and stadia or by plane-table methods in which he knows that the features "under

Note.—Written discussion of this paper will be closed in **April, 1932,** *Proceedings.*

[1] Presented at the meeting of the Surveying and Mapping Division, Sacramento, Calif., April 24, 1930.

[2] Pres., Aerotopograph Corp. of America; Fairchild Aerial Surveys, Washington, D. C.

the feet" of the instrumentman or his rodmen are located with a high degree of accuracy, but other features which are of equal if not more importance are interpolated or "sketched in." It is not an exaggeration, but a truth that should be apparent to every engineer, to state that a map properly made by adequate photographic surveying methods must be more consistent throughout than one made by what may be called the old-fashioned ground-survey methods.

Of course, there are exceptions, and the use of the aerial camera can not be cited as a solution for all surveying problems. For example, the present-day limitations of the aeroplane in flying low enough to secure photographs adequate for extremely large-scale mapping seems to rule out the use of these methods for maps on scales much larger than 1 in. = 200 ft. It would be foolish to attempt to use aerial photography in a property boundary survey of an area for which the measurements of directions and lengths of the courses are required with a high degree of precision. Moreover, areas covered by dense and tall timber hiding the surface of the ground, areas in which the ground has little or no photographic contrast, and areas which are continually covered by fog, mist, or cloud shadows often make impracticable and sometimes prohibit the use of aerial photographic mapping.

Nevertheless, these and other exceptions comprise only a small part of the mapping activities which confront the engineer and in most mapping projects the aerial camera offers the engineer a new tool which will give him more help than all the other surveying and mapping instruments combined. This statement also must be qualified, for the aerial camera can not be expected to solve many problems without the aid of ground-surveying instruments, and even the most enthusiastic exponent of its use does not believe that it will ever entirely displace the transit and the plane-table.

A properly designed aerial-surveying camera may be considered as a combination of transit and plane-table. With the aid of relatively little ground-survey control and proper laboratory instruments, triangulation can be extended from the photographic data at least with tertiary accuracy sufficient for the details of most mapping projects. Photographs taken with such a camera afford a wealth of detail that could not be recorded by means of either the transit or the plane-table without an expenditure of time and effort often not justified. In fact, the writer believes that there are many inaccessible areas which can be mapped adequately by no other method than an aerial one; for instance, low-lying swamp lands or coastal areas.

Fundamental Principles of Stereoscopic Measurement

The examination and use of photographic data require a clear understanding of the principles of perspective and a knowledge of certain controlling factors, such as the position, elevation, and angle of tilt of the camera, and the positions and elevations of certain ground-control points. However, the use of such photographs without some practical mechanical means of viewing them stereoscopically is likely to lead the observer to miss much minor detail which he should have at his disposal. In fact, it is as foolish for an observer to attempt to study overlapping aerial photographs

without the use of at least some kind of simple stereoscope as it would be for a ground surveyor to attempt to eliminate the use of a protractor or a plotting scale.

The general principles underlying the stereoscopic use of photographs as practiced in all the modern methods are much the same. A detailed description of these general principles will not be given in this paper as they have been adequately demonstrated elsewhere.[3] In the simple stereoscope, two photographs of the same object or area, taken from the ends of a base line and with the camera negatives inclined to the horizontal plane by approximately the same angle, can be adjusted to form a simple plastic image of the object or area. This gives the same impression of distance and relief as is perceptible to the human eye, except that the impression of distance derived from the stereoscopic examination of a pair of photographs taken with the axis of the camera in a horizontal position, or of relief derived from examination of a pair taken with the camera axis in a vertical position, is, of course, only relative. For mapping purposes, certain absolute control data are required, such as the positions and elevations of the camera stations or the positions and elevations of at least three points in the field of view. With adequate ground-control data and with photographs viewed in a properly designed measuring stereoscope, the positions and elevations of all the features shown in common on the stereoscopic pair can be determined.

Of course, the problem is not as simple as the foregoing statement may indicate, and both the theory and practice of the stereoscopic operations vary with the nature of the problem and the accuracy desired. One of the best discussions of the stereoscopic use of photographs published in English is that by Capt. M. Hotine, R. E. The following description is prepared from Captain Hotine's paper.[4]

To demonstrate the stereoscopic fusion of two images of the same object as viewed from two different positions, hold a sheet of cardboard between the two dots in Fig. 1, so that one dot only is seen by one eye. The other dot

FIG. 1.—STEREOSCOPIC FUSION OF ONE PAIR OF IMAGES

will be hidden by the cardboard and will be in the view of the other eye. Place the head opposite the dots in the "square on" reading position, but not too close to the dots, and attempt to look past the dots and through the paper. If any difficulty is experienced in performing this feat, make two similar marks on a window pane and look actually beyond them to some object outside. The two dots will appear to move inward and will finally come together,

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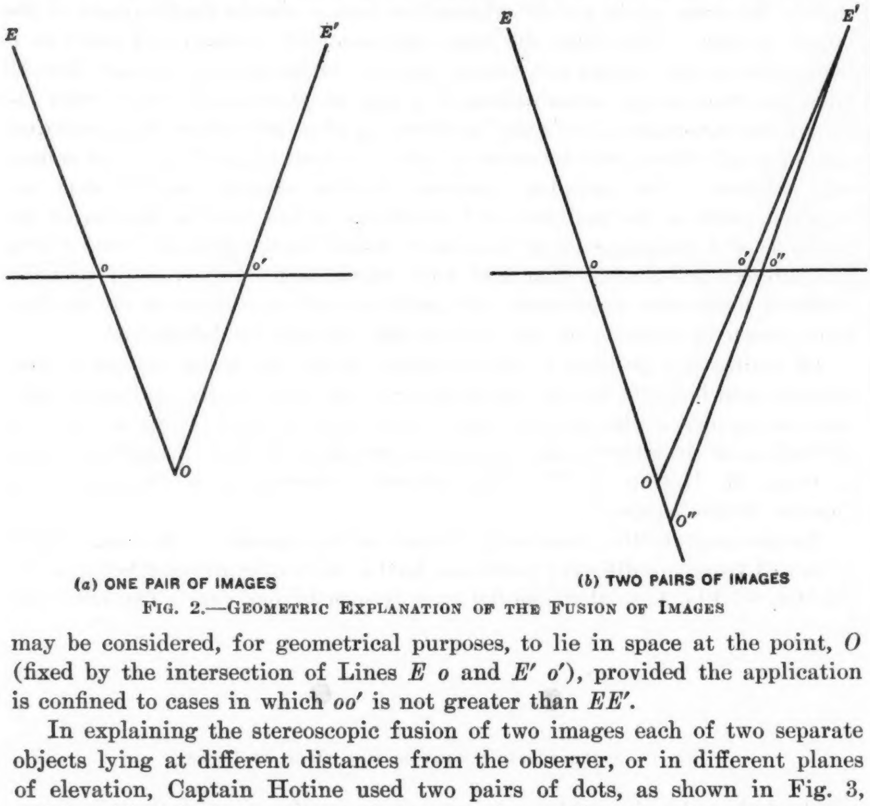
[3] "Aeroplane Topographic Surveys," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), pp. 627–655; also, *Bulletin* 788, U. S. Geological Survey, pp. 384–398.

[4] *Professional Paper No. 4*, British Air Survey Committee; see, also, "Surveying from Air Photographs," by Capt. M. Hotine, pub. by Richard R. Smith, N. Y.

appearing as a single dot in stereoscopic fusion. If this dot appears blurred maintain it in fusion by an effort of will and look back at the paper, thus bringing the fused image into visual focus.

By means of the foregoing procedure one is able to appreciate the existence of only one dot where two actually exist. The question of where the third dot is, in relation to the original two, is answered by the geometrical principle of ortho-stereoscopy. In Fig. 2(*a*) let Points *o* and *o′* be the actual positions of the two dots and Points *E* and *E′*, the positions of the two eyes. Then, according to the principle of ortho-stereoscopy, the single fused dot



(*a*) ONE PAIR OF IMAGES (*b*) TWO PAIRS OF IMAGES

FIG. 2.—GEOMETRIC EXPLANATION OF THE FUSION OF IMAGES

may be considered, for geometrical purposes, to lie in space at the point, *O* (fixed by the intersection of Lines *E o* and *E′ o′*), provided the application is confined to cases in which *oo′* is not greater than *EE′*.

In explaining the stereoscopic fusion of two images each of two separate objects lying at different distances from the observer, or in different planes of elevation, Captain Hotine used two pairs of dots, as shown in Fig. 3, which, with a little practice and the use of a cardboard strip, can be fused into two separate dots, provided the two horizontal pairs are not separated too far from each other.

If proper fusion is obtained, the observer will at once see that the upper fused dot is appreciably nearer than the lower. Captain Hotine explains[4] this with the statement that there is a binocular parallax between the two images equal to the distance, *o′ o″*, on Fig. 2 (*b*), which is the difference in the separation of the component dots. As in the case of normal binocular vision, it is the existence of this parallax that gives rise to a perception of relative distance.

In Fig. 2 (*b*), the angle, *EOE′*, is the "parallactic angle" or the "angle of convergence" of the visual directions from the eyes to the fused image, *O*. Similarly, the angle, *EO″E′*, is the convergence to the image, *O′*, formed by the fusion of the two corresponding points, *o* and *o″*. The angle, *OE′ O″*, represents the difference of these two angles of convergence, and corresponds

FIG. 3.—STEREOSCOPIC FUSION OF TWO PAIRS OF IMAGES

to the parallax, *o′ o″*. Consequently, the existence of such a difference of convergence is associated with a perception of relative distances.

The writer has repeated these illustrations almost in Captain Hotine's identical words because they express so clearly and so simply the fundamental principles of stereoscopic measurement of distance and depth and because it is important actually to see the fused image of one pair of dots floating in space above the fused image of the other pair. It is then evident that, if one can measure the difference in elevation between the fused dots with precision and have some means of recording that difference, an entirely new field in map construction has been opened.

MEASURING STEREOSCOPES

From the stereoscopic impression of binocular parallax, which is a factor that can be measured, the observer is able to evaluate his estimation of relative depths in measurable terms, provided he has at his disposal an adequate measuring stereoscope. If the aerial photograph be considered as made up of a large number of dots, and two overlapping vertical photographs are viewed stereoscopically, it is obvious that the observer needs only the relative elevations of a sufficient number of dots or features to control his measurements in order to determine the elevations of all the dots or features represented in the stereoscopic pair. It is equally obvious that if all the elevations can be determined, half the problem of drawing contour lines is solved, and there remains only the need of some device to draw these contour lines to some definite and uniform scale.

In all measuring stereoscopes, there are two index marks (crosses, circles, triangles, etc.), one in each binocular telescope, and the stereoscopic fusion of these two marks forms a pointer which is commonly called a floating mark. This takes the place of the cross-hairs in a transit telescope, and enables the operator to measure and trace in the third dimension.

Ability to "see stereoscopically" requires both training and practice. Some possess remarkable stereoscopic power, but the majority of engineers have had neither the training nor the time to investigate the subject. The exercise, in so far as the eyes are concerned, is not harmful, and experience has shown that although progress may seem to be quite slow at the start, persistent

effort will result in increasing ability to obtain stereoscopic fusion easily and without effort. This fusion is difficult, if not impossible, to obtain if the observer is at all under the influence of alcoholic drink. Jocular references to the sensation of "seeing double," therefore, may have some basis in fact; however, it should be stated that the effect is only temporary.

After ability to see clearly and rapidly with an ordinary stereoscope has been acquired, practice will be necessary to accustom oneself to the use of the floating mark. This pointer provides a reference mark by means of which the varying parallax caused by changes in the relief of the terrain can be made apparent. The fused image, or pointer, such as is herein mentioned, will appear to float in space relative to the landscape. By moving the plate-holders, it may be placed farther away or closer to the image of the ground by varying the difference in parallax existing between the mark and the image of the ground, shown as a plastic model in the fused aerial photographs.

It is not always possible to secure immediately a perfect stereoscopic effect in a measuring instrument having a floating mark or pointer. A person who can obtain a good plastic effect from an ordinary stereoscopic pair of photographs often becomes confused while observing the image moving behind a fixed pointer. The untrained eye wanders from image to pointer, and when the apparent level of the pointer is changed, has difficulty in restoring the stereoscopic effect. This lack of co-ordination may be gradually overcome by training and practice, and when finally secured, remains indefinitely. The time required for this training varies with different individuals, but it is safe to say that two weeks' application would be sufficient for one having normal sight and ability, to learn to see well with the ordinary stereoscope.

There are several different types of measuring stereoscopes, ranging from instruments of the stereo-comparator type—which permit the measurement of differences of elevation, but require independent plotting—to instruments which combine the functions of measuring and plotting into one operation.

A stereo-comparator is an instrument aptly described by its name. A rather crude description is an instrument which accommodates a pair of stereoscopic views of the same object or area taken in, or rectified to, the same plane and, by measuring parallactic displacements between fused images of points, permits the evaluation of the three co-ordinates of the points. This is accomplished by comparing measured parallaxes of "unknown" points with those of points the three co-ordinates of which are known. The mechanics of the operations involves the movement of one photograph toward or away from the other in the direction of the line of flight and the accurate measurement of these movements. Such instruments permit "point to point" plotting in a three-dimensional system. Contour maps can be made by their use, but the process is slow and laborious. Instruments of this type are made by most firms that specialize in photogrammetrical instruments (see Figs. 4 and 5). An interesting but somewhat theoretical discussion of point plotting by an instrument of this type has been issued by Syracuse University.[5]

[5] "Topographic Mapping from Aerial Photographs by Measurements with the Stereo-comparator," by Prof. Earl Church, *Bulletin*, Syracuse Univ., June, 1930.

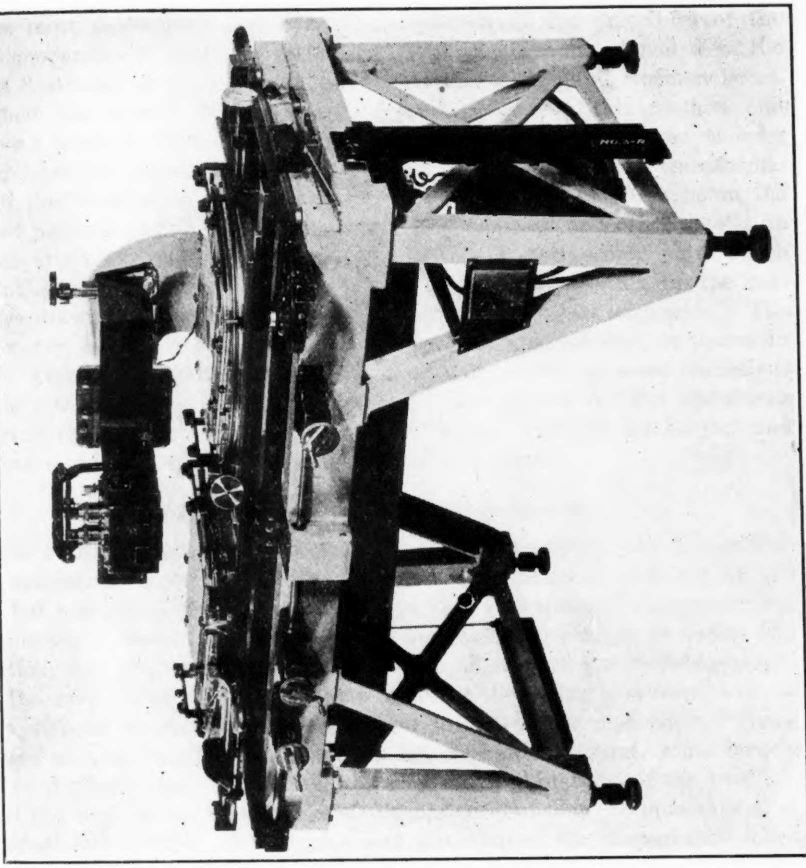


FIG. 5.—AN AMERICAN TYPE OF STEREOMETER

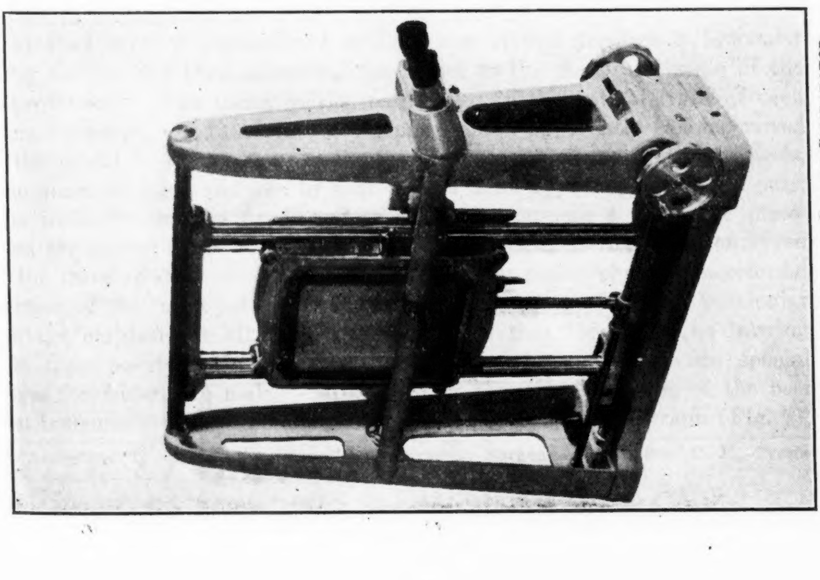


FIG. 4.—VERTICAL TYPE OF STEREO-COMPARATOR, EUROPEAN DESIGN

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The most noteworthy and effective adaptation of the principles of the stereo-comparator to mapping by use of aerial photographs perfected in the United States is the process developed by Messrs. Arthur and Norman Brock and their associates.[6] The essential difference between this method and European methods is that it uses three different instruments and thereby accomplishes the map drawing in three separate operations: First, transformation of the tilted aerial negatives to equivalent vertical photographs in the form of positive plates; second, the drawing of contours and other details on a transparent medium placed over one plate of a stereoscopic pair, which drawing is still in perspective; and, third, the transformation of the perspective drawing to an orthographic projection to the proper map scale. The stereometer, by which the perspective drawing is accomplished, is shown in Fig. 5. One of the particular advantages claimed for the separate operations of this method is the ability to use more instruments for the operations requiring the most time, thereby eliminating the "neck of the bottle" and effecting more uniform flow of work through the plant.

Measuring and Drawing Stereoscopes

Most of the European instruments accomplish the operations of rectification, measuring, plotting, and drawing in one instrument, operated by one man, but sometimes using an assistant to help in making the instrumental adjustments. Most of these instruments are somewhat similar in design and operation, and most of them provide means of actuating a drawing pencil over the map sheet so that the map features, including contours, will be drawn directly in orthographic projection to the proper map scale. Some of them accomplish the results largely by mechanical means, some largely by optical means, but most of them by varying combinations of the two.

All the best known European instruments accommodate oblique as well as horizontal and vertical photographs, and are adapted for photographs taken from the air or from the ground. The essential operations involve the placing of a pair of overlapping plates in a pair of plate-holders and adjusting them so that they form a stereoscopic model when viewed through a binocular viewing device, and then adjusting the model to the plane and scale of the map projection. Two index marks, one placed in the optical train of each viewing telescope, are fused into one image which apparently can be moved over the model in the three co-ordinates of space by means of three wheels, two actuated by hand and one by foot. This is an apparent movement only, as the index marks are fixed, and what really happens is that the plate-holders are moved so as to give this result. Plotting devices are connected with the three space movements so that a drawing pencil plots the horizontal projection of the fused pointer as it is moved over the model. The particular advantage claimed for all these instruments is that they are line-drawing rather than point-plotting instruments. Each of them has some special features for which the maker claims specific advantages. Some of the best known instruments are the stereoplanograph (Fig. 6), the autograph (Fig. 7),

[6] "Aeroplane Topographic Surveys," by George T. Bergen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 627.

and the aerocartograph (Fig. 8). Instruments of this type have also been developed in England, France, and Italy.

This paper covers in detail only one of the several instruments used in photo-topographic mapping. It is left to discussers to develop marked differences between this type and others.

Most of the early efforts to perfect stereoscopic measuring and plotting instruments were directed to the use of photographs taken from ground stations by a photo-theodolite. In Germany, R. Hugershoff, M. Am. Soc. C. E., Professor of Geodesy in the Technical Hochschule at Dresden, has been exceedingly active in the development of photogrammetrical instruments. His first "automatic" measuring and plotting instrument was known as the autocartograph (Fig. 9). This instrument was adapted for use with photographs taken from the ground and from the air, the former both horizontal and oblique, and the latter both oblique and vertical. The instrument was cumbersome and expensive, but performed its function with a remarkably high degree of accuracy. This instrument and the stereoplanograph (Fig. 6), showed an exceedingly high degree of accuracy in drawing large-scale maps with contour intervals as small as $\frac{1}{2}$ m.[7] Both require the determination by ground-survey methods of at least three elevations for each stereoscopic pair of photographs. Dr. Hugershoff bent his efforts to the development of an instrument that would eliminate some of the ground-survey operations by permitting the extension of horizontal and vertical triangulation from the photographic data, and that would be smaller in size and cheaper in cost. The results are the aerocartograph (Fig. 8), and the aerosimplex (Fig. 10) which is designed for small-scale reconnaissance mapping.

The Aerocartograph

The aerocartograph (Figs. 8 and 11) is a combination viewing, measuring, plotting, and drawing instrument. By an ingenious arrangement it also permits the carrying of horizontal positions and elevations throughout a series of overlapping photographs without intervening ground control.

The general theory of the aerocartograph is based on using two overlapping photographs and observing the two images of any point through two telescopes so as to determine the position in space of that point by means of the intersection of the lines of collimation of the two telescopes. The photographs may be positives or negatives and on glass or film; but if they are positives they are usually made by the contact process, as any tilt or scale difference in the negative is rectified in the instrumental adjustments. The overlapping pair is placed in two plate-holders in the same position in respect to each other and to the ground as they had at the moment of exposure. The plate-holders are equipped with matched lenses of the same focal length and other optical properties as the camera lens with which the photographs were taken.

The observer sits at the instrument and views the two photographic images of a point with two telescopes just as if he observed the actual point once

[7] "Ueber die Prüfung der Genauigkeit der aus Luftlichtbildern Hergestellten Topographischen Grundkarte 1:5 000 von Amrun und ihre Wirtschaftlichkeit."

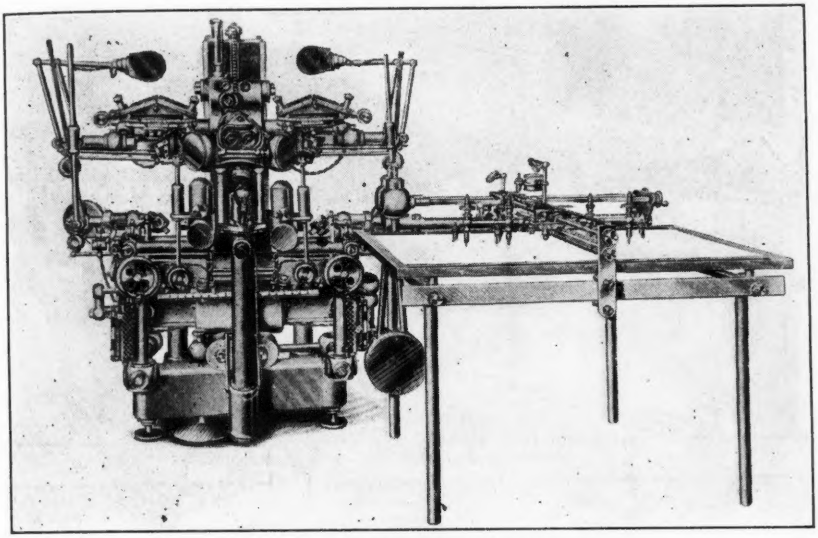

FIG. 6.—THE STEREOPLANOGRAPH

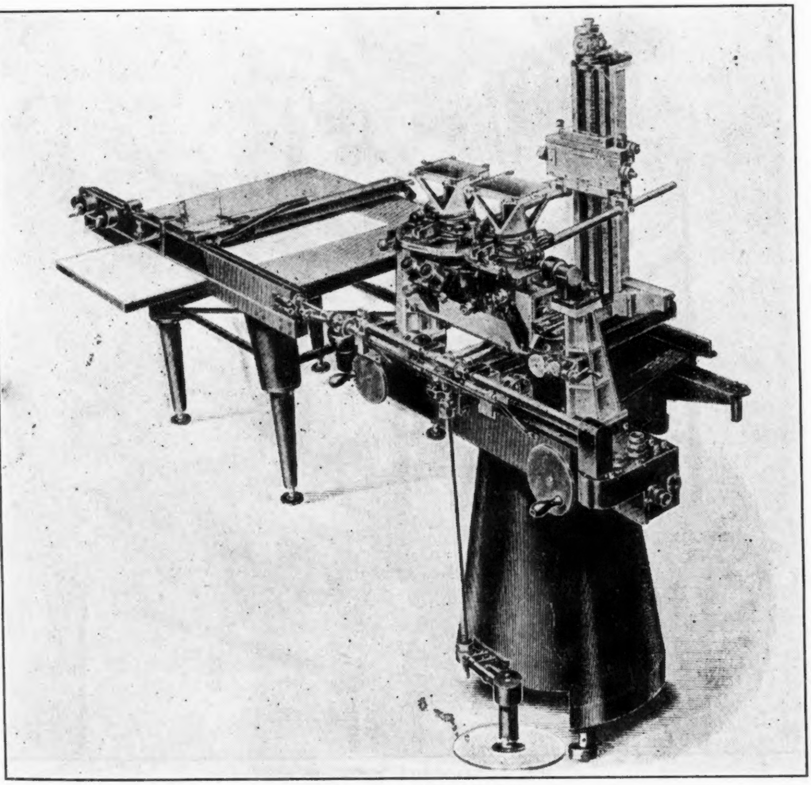

FIG. 7.—THE AUTOGRAPH

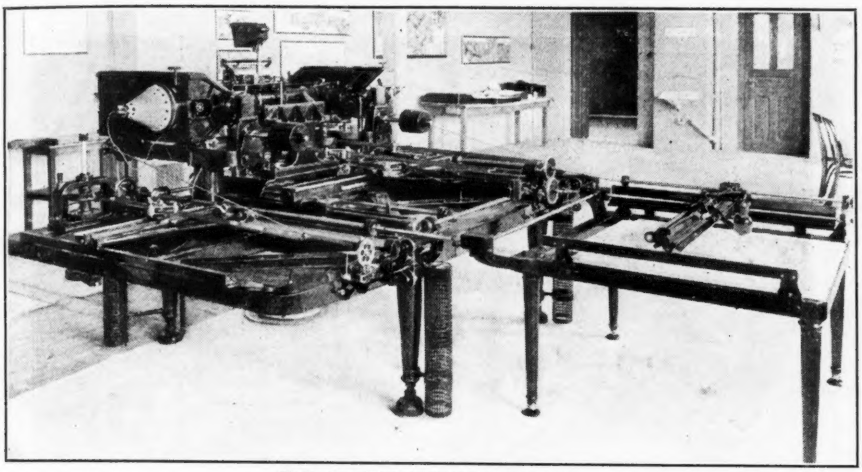

FIG. 8.—THE AEROCARTOGRAPH

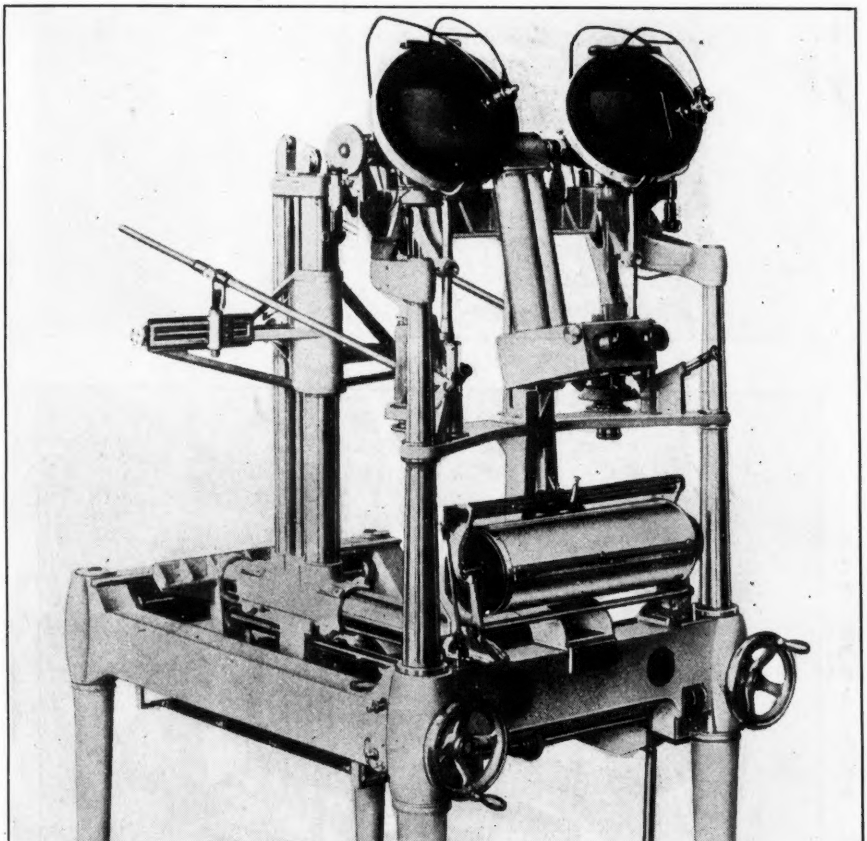

FIG. 9.—THE AUTOCARTOGRAPH

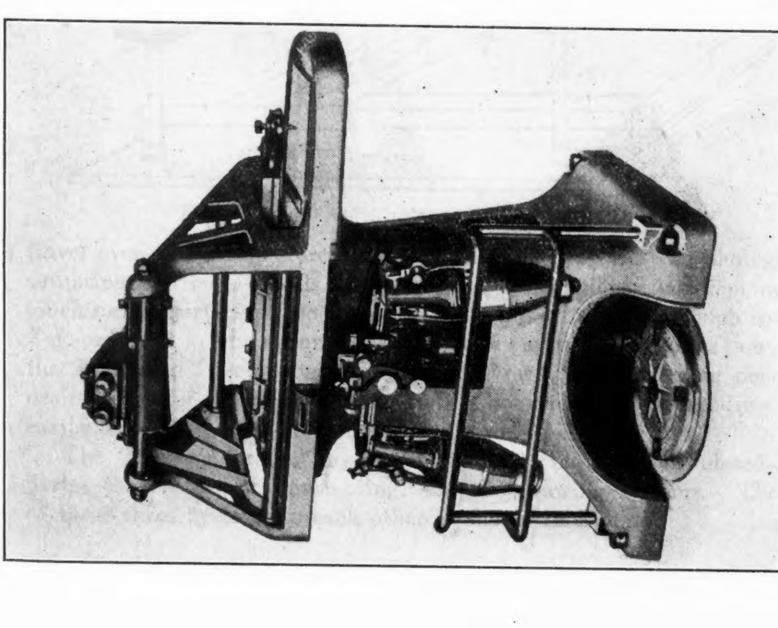

FIG. 10.—THE AEROSIMPLEX

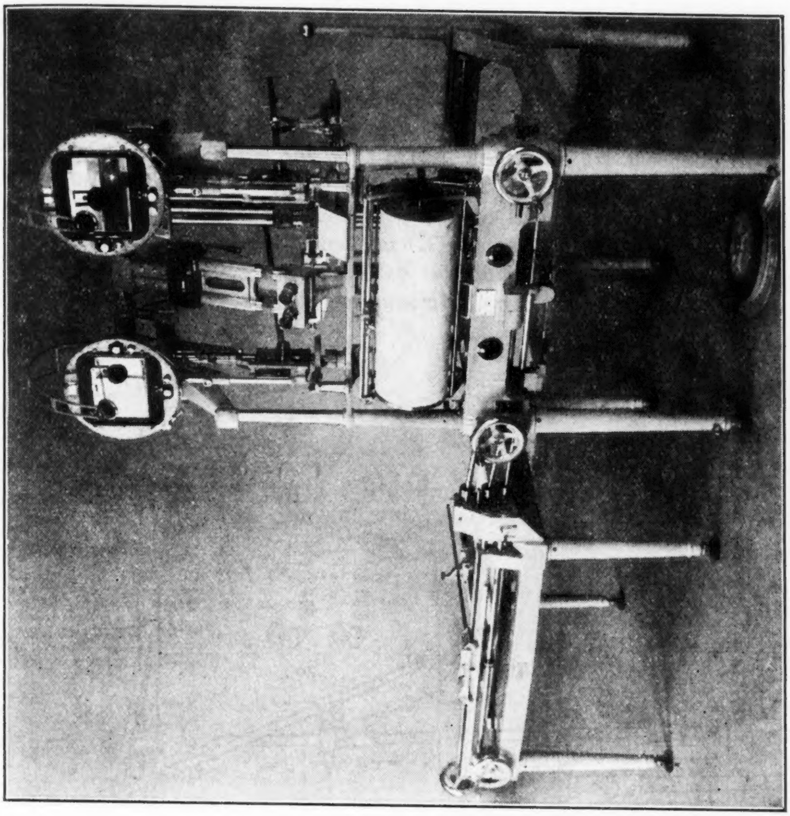

FIG. 11.—THE AEROCARTOGRAPH WITH CO-ORDINATOR

with a telescope from the first position of the aerial camera and again with the telescope from the second position of the camera. In setting the index marks or "cross-wires" in the two telescopes on the two images of the same point, certain movements are transmitted to the sight bars which place them in positions that correspond to lines of sight to that point from the two positions of the camera. These positions of the sight bars are mechanically projected on horizontal and vertical planes so as to place the drawing pencil in the plotting device at the intersection of the two horizontal (X and Y) components and to record the vertical (Z) element on the vertical component.

The stereoscopic fusion of the two index marks or cross-hairs forms a "floating mark," as previously described, which can be made apparently to

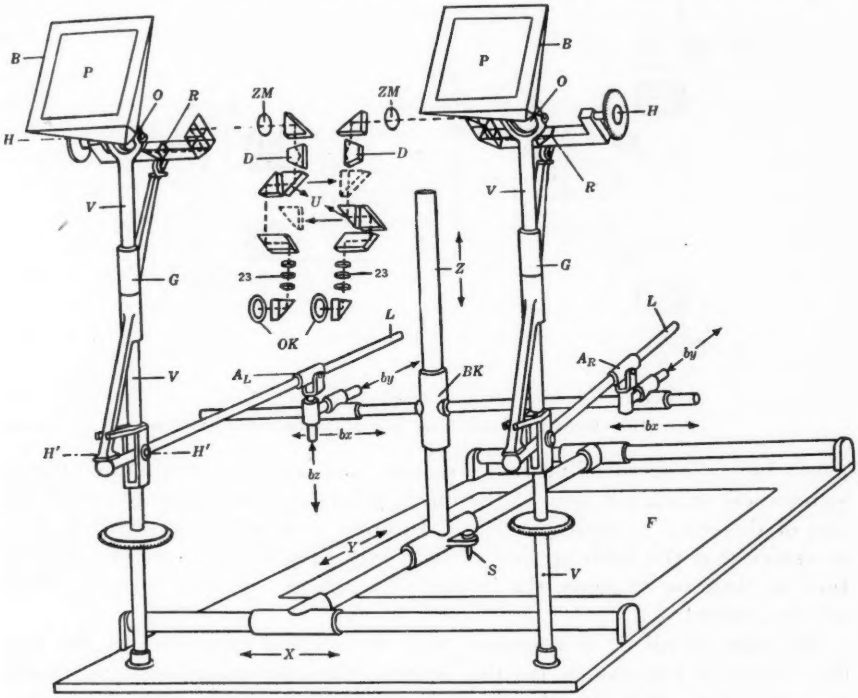

FIG. 12.—SCHEMATIC DRAWING OF THE AEROCARTOGRAPH

travel over the plastic stereoscopic model formed by the two photographs, by actuating two hand-wheels in the X and Y-directions, and kept apparently touching the surface of the model by actuating a foot-wheel which controls the Z-co-ordinate. An accurately graduated system of scales on each of the X, Y, and Z components adjusts the trace of the drawing pencil to the desired scale of the map and permits the conversion of the reading on the Z component into elevation in the unit desired—meters or feet.

The "aerocartograph," as an instrument, may be considered in three parts—the optical, the measuring, and the drawing systems. The relation of these three systems to each other is shown in Fig. 12.

The Optical System.—This consists of a double telescope (Fig. 13), which enables the operator sitting in front of the instrument to observe at the same time two corresponding images of the same object without forcing or straining his eyes. This is possible only when the eye-pieces, *OK*, correspond exactly with the horizontal distance between the pupils of the observer's eyes. Facilities for adjusting the eye-pieces to meet this condition are provided by the screws (25) which permit their separation to fit the pupillary distance, and the screws (26) which permit the movement of either one up or down. Each eye-piece can be focused separately by turning the focusing rings (24). To obtain the highest possible accuracy in stereoscopic setting of the index

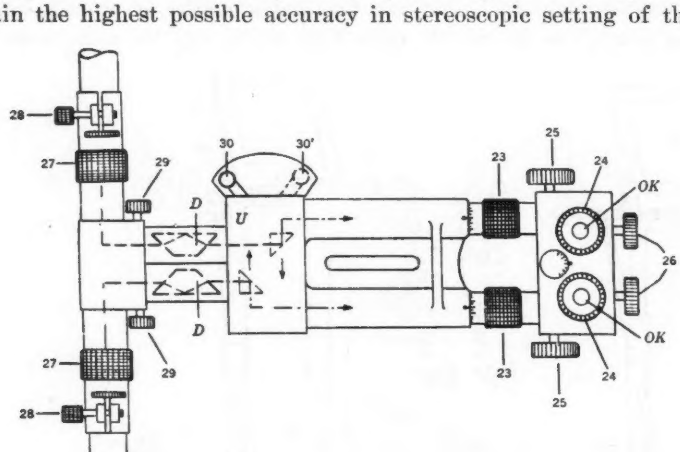

FIG. 13.—DETAIL DRAWING OF THE MAIN OPTICAL SYSTEM

marks on the corresponding image points, it is necessary to obtain complete stereoscopic effect, not only of the image points that are being observed, but also of the entire stereoscopic field. In order to meet these conditions it is necessary that the fields of view of both eyes have the same optical orientation, so that, for example, the images of a straight road in both eye-pieces will be parallel.

In other words, it is necessary that the vertical projection of the base line, which in this case is the line between the two camera stations, should be as nearly parallel with the observer's eye-base as possible. This is accomplished by adjusting the diagonal reflecting prisms, *D*, which serve to turn the fields of view of each eye-piece in the desired direction. It is also necessary that the fields of view in the two eye-pieces be brought optically to the same scale. This is accomplished by turning the image focusing sleeves (27). Finally, it is desirable to bring the stereoscopic field of view optically to as large a scale as possible, taking into consideration the size of the grains of the emulsion on the plates and the sharpness of the images. This is accomplished by turning the two magnification sleeves (23) to the same setting. These permit changing the magnification from two to four and one-half times. However, if the magnification is increased the area covered by the field is decreased.

Besides fulfilling all conditions required by modern stereoscopic practice, the optical system contains a special installation, *U*, which, by means of a lever (30–30′), permits the switching of a prism in each telescope to a corresponding position in the other telescope so that the right-hand plate can be viewed with the left eye, and *vice versa.* This switching of prisms has two advantages: First, the changing from normal stereoscopic vision to pseudo-stereoscopic observation, in which mountains appear as valleys and *vice versa,* and trees appear to be growing into the ground instead of above the ground, permitting duplicate measurement of their height, once in normal and once in pseudo-position; and, second, the changing of only one plate at a time in a sequence of overlapping photographs, so that after the operator has completed his observations on Plates 1 and 2 in the series, Plate 1 can be removed and Plate 3 inserted in its place and adjusted by images alone to Plate 2, so as to form a second stereoscopic model properly conformed to the map projection. The last mentioned operation makes it possible to carry horizontal and vertical control from one photograph to another without any ground control except in the first and last plate pair of the series.

The Measuring System.—The recording of directions in space by the aerocartograph is analogous with the measurement of these directions by a transit, but instead of sighting the natural object with a transit, the aerocartograph operator observes the photographic image of this object. In order to locate a point in space by a transit, two directions in space must be measured, one from each end of the base, involving the use of two transits or one transit placed, successively, at each base end station (aerial or ground camera stations). The aerocartograph operator locates this point in space by sighting two photographic images of the same point, one taken from each camera station, by a binocular telescope.

The photographic images are contained in the diapositive plates, *P*, in the plate-holders, *B* (Fig. 12). These plate-holders are optical substitutes for the exposure camera and the diapositive plates are reproductions of the camera negatives in the two successive positions of the camera. The angular measurements are made through the lenses, *O*, which are duplicates of the lens in the exposure camera.

As an ordinary transit telescope is turned on a horizontal and vertical axis when directed at any point in space, it is necessary to have the same movements, or substitutes for them, in the aerocartograph in order to cover the entire field of view in a stereoscopic pair of photographic plates. In the operation of a transit the direction in space of the line of collimation is resolved into two components which are recorded as horizontal and vertical angles. In the same manner the line of collimation of each observing telescope in the aerocartograph, when directed at the photographic image of a point, is resolved into two components.

However, the viewing of the photographic image of the landscape instead of the landscape itself permits a substantial simplification in the construction and movements of the sighting telescopes. Instead of rotating the telescope on its vertical axis in order to measure horizontal angles, the telescope is kept in a stationary position and the plate-holder is rotated on its vertical

axis, *V*. Instead of inclining the telescope on its horizontal axis in order to measure vertical angles, a set of reflecting prisms, *R*, is rotated on a horizontal axis, *H–H*, in front of the objective lens of the telescope. The use of a fixed and stationary binocular telescope simplifies the simultaneous operation of two telescopes and permits stereoscopic examination of two corresponding images of the same natural objects. The optical intersection of the two lines of sight determines the location of an object in its three co-ordinates in space. This imaginary intersection, of course, must be rectified to the scale of the co-ordinate system.

The mechanical expressions of the two lines of collimation are the guide rods, *L*, which act like two transit telescopes. These move up and down on their horizontal axes, *H′–H′*, and rotate about the vertical plate-holder axis, *V*. An azimuthal change of their direction, with respect to the co-ordinate system of the instrument, causes the plate-holders to turn simultaneously through the same horizontal angle. A change in the inclination of the guide rods is transmitted through the sliding members, *G*, to the revolving reflecting prisms, *R*. This is accomplished with angular precision. Thus, the guide rods, when once made parallel to the lines of collimation of the corresponding telescopes, will be the true representation of the lines of collimation of the telescopes and will permit directing them toward any point in the photographic field of view. The line of sight in each telescope is defined by an index mark, *Z M*, which corresponds to the cross-wire in an ordinary telescope. These index marks are engraved on the inside face of a lens as indicated in Fig. 12. The rays of light originating from image points on the photographic plates are brought into coincidence with these marks in their plane. By forming this coincidence in this place it is possible to avoid measuring errors which would have been introduced if the plane of coincidence had been in other parts of the optical system. Binocular observation with both telescopes properly adjusted brings these two index marks into fusion and produces a single pointer or floating mark which seems to touch the object sighted.

Different lenses used in exposure cameras, although of the same specifications, may have slightly different focal lengths from those in the plate-holders. This difference of focal length must be compensated for in the plate-holders by changing the position of the plates with respect to the lenses. This is accomplished by special focusing rings on the plate-holders.

The horizontal position of an image point is recorded in the drawing system either graphically or on the two horizontal co-ordinate scales. The elevation is recorded on the vertical co-ordinate scale and also on a rotating recorder something like a speedometer placed at the right side on the front of the instrument so that the operator can observe the elevation reading without moving from his seat. The graduations on all the recording scales are in millimeters and can be read to tenths by means of verniers. Conversion of these readings to actual dimensions in space in the units desired is simple and depends on the map scale.

The Drawing System.—This consists of a three-dimensional co-ordinate system (Fig. 14) functioning with the guide-rods so as to draw a horizontal

projection of the photographic images to the map scale desired. In comprehending this system, one must keep in mind that the instrument is designed for use either with vertical (or oblique) photographs taken from the air, or with horizontal (or oblique) photographs taken from the ground. (It is understood that references to vertical and horizontal photographs mean photographs taken with the optical axis of the camera in a vertical or hori-

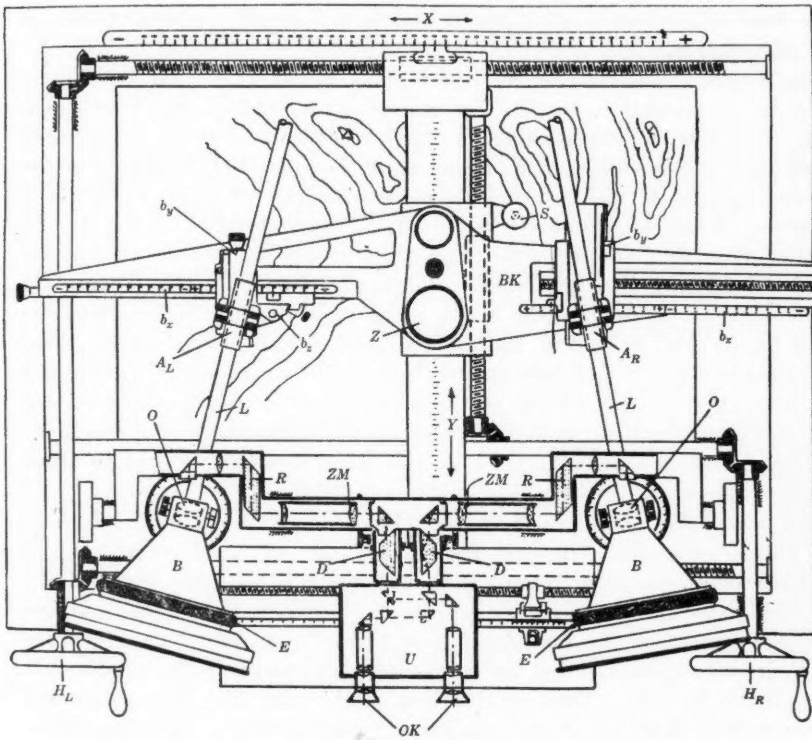

FIG. 14.—THE AEROCARTOGRAPH AS SEEN FROM ABOVE

zontal position.) One must also keep in mind that the plane of the drawing-board in the base of the instrument is approximately at right angles to the plane of the plates in the plate-holders so that the axes of co-ordinates in the drawing system differ in drawing from vertical and from horizontal photographs. In the case of horizontal photographs, the Z-co-ordinate, as shown on the diagrams in Figs. 12 and 14, is the elevation element, while in the case of vertical photographs the Y-co-ordinate is the elevation element.

It is easier first to consider the use of the instrument with horizontal photographs taken from ground stations, because these photographs are really taken in a plane at right angles to the plane of the map to be made, which planes correspond to the planes of the plates and the drawing-board in the instrument, respectively. It is then easy to understand how, by a simple shifting of gears, the movements are adapted to the proper co-ordinates in

drawing from vertical aerial photographs. The axes of the drawing-board, *F*, are the *X* and *Y*-axes of the instrument. The *X*-axis is parallel to the observer's eye-base in a vertical plane in the case of terrestrial photographs, and in a horizontal plane in case of aerial photographs. The *Y*-axis is at right angles to the eye-base in a horizontal plane in the case of terrestrial photographs and in a vertical plane in case of aerial photographs. The *Z*-axis is perpendicular to the surface of the drawing-board in a vertical plane in the case of terrestrial photographs and in a horizontal plane in case of aerial photographs.

The *X*-axis is constructed as two parallel tracks on which the *Y*-axis can be moved parallel to itself. The *Z*-axis is attached to the *Y*-axis and, therefore, can be moved parallel to itself. On this *Z*-axis are mounted the two arms of the column, *B K*, which move the guide-rods by means of the guide-rod sleeves, A_L and A_R, and these move on the left side in the three dimensions of the co-ordinate system and on the right side in the *X* and *Y*-axes of the co-ordinate system. This construction permits the representation of the distance in space between the two camera stations as expressed on the scale of the map to be made. The axes of this base system are marked *bx, by,* and *bz* in Fig. 12, and the vernier scales by which the three components can be set and adjusted to the map scale are marked b_x, b_y, and b_z, in Fig. 14. The movements of the base system in the direction of the *X, Y,* and *Z*-axes can be made by means of two hand-wheels and one foot-wheel, all of which can be reached by the observer from his seat in front of the instrument.

With terrestrial photographs in the plate-holders, the moving of these three wheels gives the following results. If the left-hand wheel, H_L, is turned clockwise, the base system is moved along the *X*-axis from left to right. This changes the direction of the guide-rods with reference to the co-ordinate system of the instrument and turns the plate-holders clockwise about their vertical axes so that the pointer seems to travel in the stereoscopic field from left to right. Turning the right-hand wheel, H_R, clockwise moves the base system along the *Y*-axis toward the front of the instrument. This causes the guide-rods to converge so that the intersection of their directions is closer to the eye-base and the pointer seems to move to the foreground of the stereoscopic field. Turning the foot-wheel clockwise moves the column, *B K,* downward along the *Z*-axis, tilts the guide-rods, and raises the intersection of their directions, so that the pointer is moved toward the upper edge of the stereoscopic field.

These three movements, all being translated to the guide-rods, are so interrelated that one affects the others, and they must be made simultaneously. By means of these movements the pointer can be traced in any direction over the model and kept apparently touching the surface at all times. If the drawing pencil, *S,* is connected with the sliding collar at the base of the *Z*-component, it will plot on the drawing-board the horizontal projection of each optically intersected point or of the trace of the pointer over the stereoscopic model. The vertical component can then be read on the graduated

scale on the Z-column, in millimeters and decimal parts, which can readily be transposed to elevations above the datum plane in the units desired — feet or meters.

If the foot-wheel is turned so that the reading on the Z-co-ordinate scale represents a certain contour and is left in that position so that the elevation reading will always be the same, the pointer can be forced over the model and apparently kept in contact with its surface by means of the X and Y-movements alone. The drawing pencil then traces the horizontal projection of a continuous series of points of the same elevation which must be the proper contour line.

With vertical aerial photographs in the plate-holders the relation of the measuring and drawing systems to the optical system must be changed so that the co-ordinates will conform to the length, breadth, and depth of the stereoscopic field. In other words, the plane of the plate-holders should be made horizontal, or the plane of the drawing-board should be made vertical. As it would be mechanically impracticable to make either of these changes, the simplest solution is to to select the X and Z-co-ordinates of the instrument as a theoretical map plane corresponding to a vertical drawing-board. In order to accomplish this, a second Z-spindle (31) is thrown into gear so that it is driven by the right-hand wheel and the same gear-switching transmits the movement of the foot-wheel to the Y-spindle. The drawing pencil is then connected directly to the gear which drives the second Z-spindle so that it receives the same movement from the right-hand wheel as the second Z-spindle. The theoretical vertical map plane has thus been turned 90° and the horizontal drawing-board, *F*, now serves for drawing from vertical aerial photographs.

With this arrangement, if the left-hand wheel is turned clockwise, the result is the same as with terrestrial photographs, and the pointer seems to travel in the stereoscopic field from left to right parallel to the line of flight. If the right-hand wheel is turned clockwise, the column, *B K*, is moved upward along the Z-axis, tilting the guide-rods so that the pointer is moved toward the lower edge of the stereoscopic field — in the same plane as the X-movement, but at right angles to it, the two movements being in the horizontal plane in space. The drawing pencil, however, moves in the direction of the Y-axis toward the front of the instrument. Turning the foot-wheel clockwise moves the base system along the Y-axis toward the back of the instrument. This causes the guide-rods to diverge so that the intersection of their directions is farther away from the eye-base and the pointer seems to move downward or into the ground. The drawing pencil will then plot in the X and Y-co-ordinates the horizontal trace of the movement of the pointer over the model, and the vertical component or elevation can be read on the graduated scale on the Y-axis.

The aerocartograph has two other drawing surfaces so that, if desired, three maps can be drawn at the same time on the same or on different scales. The drawing drum shown in Fig. 8 is placed on the front side of the instrument just below the observer's eyes. This can be covered by tracing paper and, by an ingenious gear arrangement, the same movements in the plane

of the map are transmitted to a second drawing pencil and a second copy of the map is made. The particular advantages of this arrangement are that the observer can see what he is drawing without moving his position, and he can remove at any time the tracing paper and compare his work with the plotted positions on a master sheet.

The third drawing surface is on a separate table known as the co-ordinatograph, which is geared to the co-ordinate spindles of the map plane and on which a third pencil traces the apparent movement of the pointer in the same way and with the same accuracy as the pencil on the base drawing-board. The particular advantages of the co-ordinatograph are that the drawing-board is more accessible and the gear ratio permits a larger change in scale from the photo-scale than can be accomplished by the drawing system in the base of the instrument. The co-ordinatograph also permits the plotting of projections and control points from known co-ordinates with more facility and accuracy than can be accomplished by the usual drafting means.

Effect of Relief.—The pointer provides a reference mark by which the varying parallax caused by the changing relief of the terrain may be made visible. It is understood, of course, that the term, "pointer," refers to the fused image of the two index marks, one in each eye-piece, which are rigidly fixed in the optical axes.

If identical photographs be used in each plate-holder, and viewed as one, there would be no stereoscopic effect, and the pointer would remain in apparent contact with the ground at all points in the area, no matter how great the relief. If a stereoscopic pair be viewed and so adjusted that the pointer coincides with the image of a hilltop, and the plate pair moved as a whole, so that the pointer covers a lower area, it will appear to float above the ground, owing to the parallax.

The fixed positions of the two index marks may be represented by the points, A and A' (Fig. 15), on a line representing the plane of coincidence where it intersects the traces of the rays from a hilltop to the camera in its successive exposure Stations 1 and 2. The two index marks are then in coincidence as seen with each eye-piece and, therefore, are fused into one image which appears to rest on the hilltop. If the left-hand wheel is turned, effectively moving the index marks in the fixed plane of coincidence to B and B', respectively, the ray from the image, X, of the lower point in the left plate intersects the plane of coincidence at B, but owing to the parallax, $B' B''$, the ray from the corresponding image, X', in the right plate does not intersect the plane at B' but at B'', and, consequently, monocular vision of the corresponding images of the lower point (seen by alternately closing the two eyes) will show a displacement between the two index marks and the two respective images of the same point on the ground. Stereoscopic vision (with each eye focused on its respective index mark) will show stereoscopic fusion of the index marks into one pointer which appears to float above the point on the ground.

This parallax, $B' B''$, is eliminated by turning the foot-plate, which diverges the plate-holders, thus bringing the pointer to rest on the ground at

the lower point. If the left-hand wheel is turned so as to move the index marks back to A and A', the parallax is shifted to the hilltop, and at that point the pointer will appear double, or deeply buried below the ground.

It will now be seen that in drawing cultural detail and drainage, the foot-plate must be used simultaneously with the hand-wheels, as roads and streams

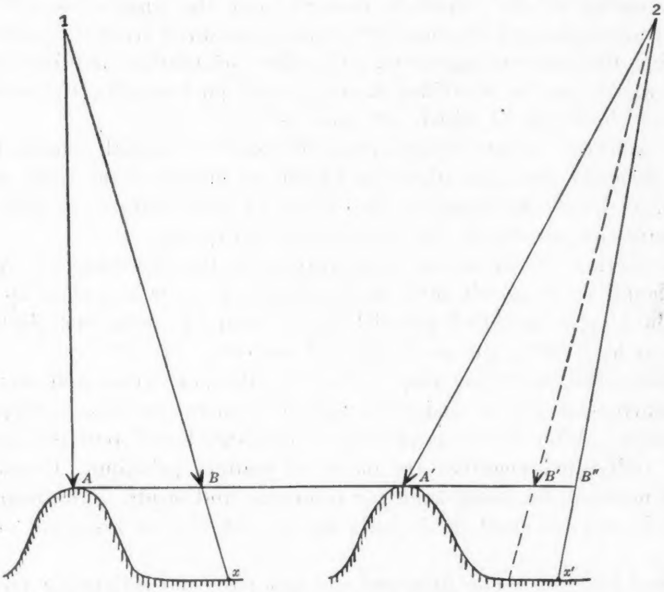

FIG. 15.—DIAGRAM OF PARALLAX CAUSED BY RELIEF

necessarily change their level. If the foot-plate is not used, it is, of course, possible to use the eye-pieces independently and cause the index marks to follow a road or a stream. The two projections of a road or a stream thus drawn would not coincide, and neither would be a true projection. While drawing contours, the foot-plate, of course, is locked.

AERIAL PHOTOGRAPHY

Since in an ordinary stereoscope, the photographs must be set in a certain position in order to get perfect stereoscopic fusion, it is evident that the same procedure must obtain in the orientation of the photographs in the plate-holders of the aerocartograph. The stereoscopic field must be oriented in a plane which coincides with the proper plane of the co-ordinate system, since it is desired actually to take measurements in this stereoscopic field and such measurement must be obtained through the co-ordinate system of the aerocartograph. The stereoscopic field must also be so oriented that the eyes of the observer will see it as if he were at the stations where the exposures were made. This is only possible when the photographs in the plate-holders have the same relative positions to the lenses in the plate-holders that the negatives had to the lenses in the exposure camera. Also, the photographs

must have a position in the drawing system of the instrument in which the relation to the horizon in the original exposure is maintained. It is essential that there be a true reproduction of the angles of the light rays as they come into the exposure camera in order to obtain the interior orientation required. The interior orientation is accomplished with the help of collimating marks in the exposure camera, and the angles formed by these collimating marks and the negative center, measured from the optical center, determine the camera constants. Exterior orientation requires only three points, which can be identified in the aerial photographs, the ground positions and elevations of which are known.

The exposure camera differs from the ordinary aerial camera in that it has an optically flat glass plate, on which are engraved the plate center and four collimating marks, one in the center of each margin, arranged so that these marks are shown on the negative when exposed.

The overlap of the aerial photographs in the direction of the line of flight should be as nearly 60% as possible and never less than 50 per cent. When the area is such that parallel flights must be made, such flights should overlap at least 10%, and preferably 20 per cent.

In the aerial surveying camera used in the aerocartograph method, uniformly shrinking film is used, although the cameras are also equipped to use glass plates. After the negatives have been developed, positive glass plates of each individual negative are made by contact printing. Contact prints are also made at the same time for reference and study. The negatives are then filed, and are used again only in case of loss or breakage of positive plates.

Ground Control.—The first and the last pair of overlapping photographs in each flight or strip are used to establish ground control. The length of such a strip or the number of photographs over which aero-triangulation can be carried safely without additional ground control depends on the scale of the map, the amount of relief, and the contour interval; but, under average conditions, it is from ten to fifteen photographs. Three points, and preferably one additional for a check, which are clearly identifiable in the overlap portion of the first two photographs are chosen, and the position and elevation of these points are determined by ground-survey methods. These three points form the vertices of a triangle and if it can have the shape of an equilateral triangle with as long sides as the overlap will permit without being too close to the edges of the photographs, the control approaches the ideal condition. A similar set of terrestrial control points should be established in the overlap of the last pair of photographs in a strip flight, although these points may be in an approximately straight line at right angles to the line of flight instead of forming a triangle. For ease of identification of selected points, it is advisable, if time permits, to photograph the area previous to the establishment of the ground control. Such points as road corners, intersecting fences, stream forks or crossings, or points easily referenced by lone trees or houses, are proper control points for aerial photographic work. The usual ground-survey triangulation points are seldom identifiable in a photograph, and it appears that, in this method of the production of

maps from aerial photographs, one must change the ground surveyor's ideas in regard to what constitutes a proper set of control points just as further investigation of the manner in which these points are located in the field may result in an entirely new technique of ground operations. Experience has shown that the aerocartograph is capable of a remarkable degree of accuracy in extending horizontal control by aero-triangulation, provided that, at least for large-scale work, the ground control is established with a high order of precision.

Mapping

After the ground-control points have been located and plotted on the map sheet, the positive plates are placed in the plate-holders of the aerocartograph, and the interior, stereoscopic, and exterior orientation is secured by optical and mechanical means in a definite routine. A considerable advantage in this method is due to the fact that in the aerocartograph instrument, there are a minimum of moving parts as well as operations or steps in the securing of the final product. When this orientation has been perfected, the operator has before him a stereoscopic picture or plastic model of the landscape. The flexibility of the method is clearly apparent, since it is now possible for him to produce at once, without intermediate operations, a map of any kind to suit any definite purpose.

To obtain the scale, it is not necessary to have ground-control points in every pair of photographs. The first pair of plates (called, for example, Plates 1 and 2) are oriented and adjusted by comparing an aerocartograph plotting of the positions and the reading of the elevations of the stereoscopic images of the ground-control points with the actual plotted positions and known elevations of those same control points. Three new picture points are then selected by images alone (and without locating them on the ground) so that they lie in the advanced edge of the stereoscopic model and will also appear on the third plate. (This is the reason for the 60% overlap). These points are selected so that the central point is about in the center of the strip and the other two points are on a line at right angles to the line of flight and near the sides of the strip of photographs.

Plate 1 is then removed from its plate-holder and Plate 3 is substituted for it and oriented and adjusted to Plate 2 by images alone, using the advanced half of the undisturbed Plate 2 in forming the stereoscopic model. The crossed vision resulting from this arrangement of plates produces a pseudo-stereoscopic effect in which hills appear as valleys and *vice versa.* This pseudo-stereoscopic effect can be corrected by shifting the prisms in the optical train so that the two plates are viewed in their normal relation and appear in proper sequence. The orientation of Plate 3 is thus accomplished without the use of any additional points located on the ground. This process is continued for all the successive plates in the series of photographs taken in any one strip flight. By means of the control points in the last pair of plates, the error of closure, if any, becomes apparent, and the entire flight is adjusted. Adequate provision is made for the correction of the difference of elevation due to the curvature of the earth, and the adjustment is made

by the use of formulas which have been proved by comparison with test strips of aerial photographs on which many ground-control points have been located for each overlapping pair of plates.

The corrected setting of each plate is then computed so that upon re-insertion in the plate-holders, the sequence of stereoscopic models can be developed in detail with the assurance that there will be perfect closure in position and elevation at the end of the strip. (In short strips, or in cases where intermediate elevations are available, the corrections are made as the work proceeds, thus avoiding duplicate insertion of the plates.) As each plate pair is inserted in the plate-holders, the map details are drawn directly from the model with the aid of the floating mark. The culture and planimetry, as well as the contours, are traced by means of the floating mark as a series of continuous lines and not as lines formed by interpolation between plotted points, as in ground-survey methods. The movements of the floating mark are plotted directly on the map sheet in the base of the instrument, or on the co-ordinatograph, these details being shown in true orthographic projection. Inasmuch as this plotting is of a continuous series of points of intersection of rays projected in a true horizontal plane, there is no pantographic principle involved in the construction and operation of the aerocartograph.

Information as to land boundaries, names, and other features not appearing on the photographs is usually determined in the field during the progress of obtaining the ground control. Any features that could not be drawn with the aerocartograph because of natural obstructions or obscure parts of photographs are added by ground-survey methods, but such operations are usually of very limited extent.

The economic value of good maps has long been apparent to the map producer. It is believed that the value of maps constructed according to proper standards is being recognized more and more by the profession, by those in authority in the development of projects involving the use of natural resources, and by the general public. This recognition will result in an increased demand for better maps, and it is belived that aerial photographic methods will serve more and more as an important aid to the map producer in meeting this demand.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

DETERMINATION OF PRINCIPAL STRESSES IN BUTTRESSES AND GRAVITY DAMS

By W. H. Holmes,[1] Assoc. M. Am. Soc. C. E.

Synopsis

This paper presents the results of a study of stresses in buttress and gravity type masonry dams and develops formulas for the analytical determination of stresses at any point on a horizontal section through dams. The structures treated in this paper are divided into three groups:

I.—Gravity dams, straight in plan.

II.—Gravity dams, arched in plan.

III.—Buttresses of flat slab or multiple-arch dams having rectangular horizontal sections that taper with the height.

It is shown that the stress equations for Groups II and III become identical with the equations for Group I when the radius is infinite and the taper is zero, respectively. Examples of several section types are given.

Introduction

The formulas derived in this paper are based on the numerical solution of principal stresses in a gravity dam, straight in plan, as presented[2] by the late William Cain, M. Am. Soc. C. E. If Professor Cain's analysis was made more general and was extended to include the vertical component of the water load on gravity dams either arched in plan, or to include tapering buttresses of flat slab and multiple-arch dams, the arithmetical work would have been extremely laborious.

Observations made by the writer and experience gained in an endeavor to obtain the principal stresses in solid and buttress dams have indicated that the ordinary method of determining the vertical normal stress at the heel and toe of high dams is not sufficient for the complete analysis of the stresses. Although engineers in the past have given thought to the existence

Note.—Written discussion on this paper will be closed in **April, 1932**, *Proceedings*.

[1] Hydr. Engr., Div. of Water Resources, State Dept. of Public Works, Los Angeles, Calif.

[2] "Stresses in Masonry Dams," *Transactions*, Am. Soc. C. E., Vol. LXIV (September, 1909), p. 208, and "Practical Designing of Retaining Walls," Seventh Edition, 1914.

of principal stresses, no great advancement in the art of the stress determinations has been made.

Vertical or inclined cracks, due to shrinkage of concrete and temperature changes, occur in large gravity dams even before any water load is applied, and an analysis of stress distribution is not complete if these cracks or inclined contraction joints are not included in the analysis. The suggested use of inclined joints in a high gravity dam arched in plan, caused the writer to develop the second group of equations for stress distribution of gravity dams arched in plan.

The analysis of the 130-ft. buttress of a multiple-arch dam indicated considerable tension at the up-stream face although the vertical normal compression stress at the point was 121 lb. per sq. in. Because the buttress cracked approximately on the line of maximum principal stress an analysis was made assuming complete severance of the structure along that line. The result of this analysis indicates the possible advantages of inclined contraction joints in this type of structure and caused the writer to derive the following formulas for the determination of internal stresses.

In the analysis used to compute the principal stresses in the structure the stresses caused by shrinkage of concrete, temperature changes, earthquake shocks, settlement of foundations, etc., and the effect of the foundation upon the distribution of stresses, are not considered. The summation of the vertical forces is equal to zero; the summation of the horizontal forces is equal to zero; and the accepted formulas, $\frac{P}{a} \pm \frac{M c}{I}$, for the determination of the vertical unit stress, are the basic equations used.

The determinations of the vertical normal stress, shearing stress, and horizontal normal stress for gravity dams straight in plan, are derived first. The first and second principal stress are computed and Professor Mohr's[3] graphical method is briefly described. The basic checks are shown and numerical solutions of the principal stresses for two triangular dams and a rectangular dam are presented in Group I.

In Group II, the vertical, horizontal, and shearing stresses and the principal stresses are derived for gravity dams arched in plan, and it is shown that with infinite radius these equations are identical to the equations of Group I. Examples of the principal stresses for a gravity dam arched in plan (assuming that the arch does not carry any of the load) is shown.

In Group III, the vertical, horizontal, and shearing stresses and the principal stresses are derived for buttresses of flat-slab or multiple-arch dams having rectangular horizontal sections that taper with the height. The stress distribution in the buttress of a multiple-arch dam has been computed with and without the introduction of an inclined joint on two assumptions: First, as a monolithic base; and, second, as containing an inclined joint completely severing the base. Some of the results of these computations are tabulated herein. Other tables and graphs have been omitted to conserve space.

[3] "Technische Mechanik," by Otto Mohr, 1914.

Notation.—For convenience, the symbols introduced by Professor Cain are used. When considered necessary these are defined in the text, as introduced, and are grouped in Appendix I for convenient reference. All stresses and loads are in terms of masonry units.

Group I.—Straight Gravity Dams

Vertical Normal Stresses.—The vertical normal stress on any plane computed by the usual formula is,

$$\frac{P}{a} \pm \frac{M\,c}{I} = p_a \text{ or } p_b \quad \text{.......................(1)}$$

in which, p_b is the vertical normal stress at the up-stream face, and p_a, the vertical normal stress at the down-stream face. (Equation (1) is often written: $\frac{P}{a} \pm \frac{6\,M}{L^2} = p_a$ or p_b).

The vertical normal stress at any point, x distance from the up-stream face equals,

$$p = p_b + \frac{(p_a - p_b)\,x}{L} \quad \text{.......................(2)}$$

Let $S = \frac{p_a - p_b}{L}$ be the rate of change of intensity of the vertical normal stress. Then, the unit vertical pressure, p, at Point x is,

$$p = p_b + S\,x \quad \text{.......................(3)}$$

The symbol, p_u, p_o, or p_d will be used for the vertical pressure at the up-stream face. The subscript, b, is omitted as p_a does not appear in the following formulas, but the subscripts, u, o, and d must be used to denote the plane on which the stress is applied.

The vertical normal stress at any point on the plane, u, is,

$$p = p_u + S_u\,x_u \quad \text{.......................(4)}$$

and the summation of the vertical stresses on the plane, u, is,

$$P_u = \sum p = p_u\,x_u + \frac{S_u\,x_u^2}{2} \quad \text{.......................(5)}$$

Let the vertical stresses on the planes, o and d, at the up-stream face be, respectively, p_o and p_d; then the summations of these vertical stresses on the middle plane and the lower plane are,

$$P_o = p_o\,x_o + \frac{S_o\,x_o^2}{2} \quad \text{.......................(6)}$$

and,

$$P_d = p_d\,x_d + \frac{S_d\,x_d^2}{2} \quad \text{.......................(7)}$$

Shearing Stresses.—The vertical shear, q_1, at any point, x, in the section between the planes, u and o, is obtained from the basic assumption that the

summation of the vertical forces is equal to zero; thus, $P_u + V_1 + W_1 - P_o - q_1 = 0$, or,

$$q_1 = P_u + V_1 + W_1 - P_o \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which, q_1 is the total vertical shear on the end of the section (see Fig. 1).

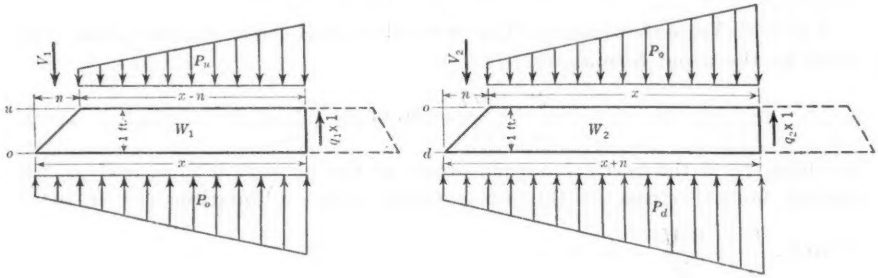

FIG. 1.

Substituting values for P_u and P_o, remembering that $x_u = x - n$, $w = 1$ = masonry unit, and $W_1 = wx - \frac{wn}{2}$, and solving Equation (8):

$$q_1 = \left(V_1 - \frac{n}{2} - p_u n + \frac{S_u n^2}{2}\right) + (p_u + 1 - p_o - S_u n)\, x + \left(\frac{S_u}{2} - \frac{S_o}{2}\right) x^2 \,..(9)$$

Let,

$$A_1 = V_1 = \frac{n}{2} - p_u n + \frac{S_u n^2}{2} \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$B_1 = p_u + 1 - p_o - S\ n \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

and,

$$C_1 = \frac{S_u}{2} - \frac{S_o}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Then,

$$q_1 = A_1 + B_1 x + C_1 x^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

The unit shear, q_1, on the end of the cross-section at x is assumed to be the average shear at a point midway between u and o on the vertical plane at x distance from the origin.

The shear on the end of the cross-section between o and d is obtained in the same manner as shown on the section between u and o. The sum of the vertical forces on the lower section is,

$$q_2 = P_o + V_2 + wx + \frac{wn}{2} - P_d = \left(V_2 + \frac{n}{2} - p_d n - S_d \frac{n_2}{2}\right)$$

$$+ x\,(p_o + 1 - p_d - S_d\, n) + x^2 \left(\frac{S_o}{2} - \frac{S_d}{2}\right) \ldots\ldots\ldots (14)$$

Let,

$$A_2 = \left(V_2 + \frac{n}{2} - p_d n - \frac{S_d n^2}{2}\right) \ldots\ldots\ldots\ldots\ldots (15)$$

$$B_2 = (p_o + 1 - p_d S_d n) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

$$C_2 = \left(\frac{S_o}{2} - \frac{S_d}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

Then,

$$q_2 = A_2 + B_2 x + C_2 x^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

The average of q_1 and q_2 is the average shear on the vertical plane at x distance from the origin, or, $q = \frac{q_1 + q_2}{2}$:

Substituting the values of q_1 and q_2 from Equations (13) and (18):

$$q = \frac{A_1 + A_2}{2} + \left(\frac{B_1 + B_2}{2}\right) x + \left(\frac{C_1 + C_2}{2}\right) x^2 \ldots\ldots\ldots (19)$$

As the horizontal shear at a point is equal in magnitude to the vertical shear at the same point, q is the horizontal shear on the plane, *o*. The summation of the horizontal shear on the plane midway between Planes *u* and *o* (see Fig. 2), is,

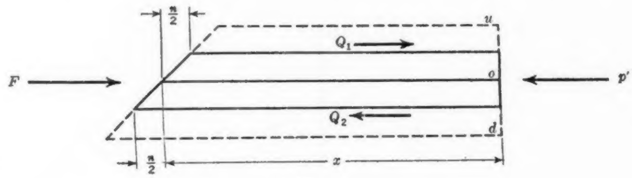

Fig. 2.

$$Q_1 = \int_{0.5n}^{x} q_1 \, dx \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

and, similarly, for the plane midway between Planes *o* and *d*,

$$Q_2 = \int_{-0.5n}^{x} q_2 \, dx \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Horizontal Normal Stresses.—The horizontal normal stress at any point, x, is obtained by the use of the basic assumption that the summation of the horizontal stresses is equal to zero, thus,

$$p' = F + Q_1 - Q_2 = F + \int_{0.5n}^{x} q_1 \, dx - \int_{-0.5n}^{x} q_2 \, dx \ldots\ldots\ldots (22)$$

Inserting the values of q_1 and q_2 from Equations (13) and (18), respectively, and integrating:

$$p' = F - \frac{n}{2}(A_1 + A_2) + \frac{n^2}{8}(B_2 - B_1) - \frac{n^3}{24}(C_1 + C_2) + x\,(A_1 - A_2)$$
$$+ \frac{x^2\,(B_1 - B_2)}{2} + \frac{x^3\,(C_1 - C_2)}{3} \ldots\ldots\ldots\ldots\ldots (23)$$

First and Second Principal Stresses.—Equations (3), (19), and (23) are the general equations necessary to determine the principal stresses at any point on the plane. After obtaining the stresses, p, p', and q, at any point, the principal stresses and their direction may be computed or obtained graphically by Mohr's diagrams. Professor Cain's method of computing the principal stress results in the derivation of the formula[4]:

$$\tan 2\,\theta = \frac{2 \tan \theta}{1 - \tan^2 \theta} = \frac{2\,q}{p - p'} \qquad (24)$$

and,

$$f = \tfrac{1}{2}\left(p + p' \pm \sqrt{(p - p')^2 + 4\,q^2}\right) \qquad (25)$$

A graphical determination of the magnitude of the first and second principal stress by the construction of a Mohr diagram[5] checks Professor Cain's derivation of stresses. To construct a Mohr diagram for stresses in one plane draw three parallel lines, a, b, and c, distant q units apart, to any convenient scale (see Fig. 3(a)). Draw a reference line, o, perpendicular

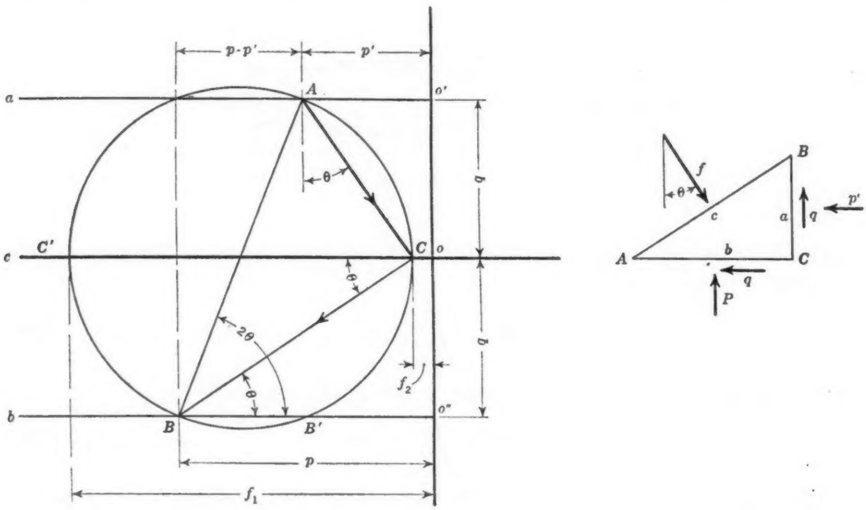

FIG. 3.

to c. Let distances to the left of o be compression and to the right, tension. Draw Ao' equal to p' and Bo'' equal to p. Draw the circle, $ACBC'$, with AB as the diameter. The distance, C', to the line, o, is the maximum or first principal stress. The direction of the first principal stress is shown by AC. It is to be noted that no tension can occur if $\dfrac{p + p'}{2}$ is greater than the radius of the circle.

[4] "Practical Designing of Retaining Walls," Seventh Edition, 1914.

[5] "Abhandlungen aus dem Gebiete der Technischen Mechanik," by Otto Mohr, 1914; also, "Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E.

Maximum Shear.—The maximum shear is equal to the radius of the circle (Fig. 3(*a*)). The diameter of the circle is equal to the difference of the two principal stresses and the radius is equal to the maximum shear, or $\frac{f_1 - f_2}{2}$.

Checks for Shearing Stress, Horizontal Normal Stress, and Principal Stresses at Up-Stream and Down-Stream Faces of the Section.—The following checks should be made of the stresses obtained from Equations (19), (23), and (25):

(1) By substituting values of x for the up-stream and down-stream faces in the equations for q and p' (Equations (19) and (23)), the following equations are obtained:

For the up-stream face:

$$q = V - pn \qquad (26)$$

and,

$$p' = F - q\,n \qquad (27)$$

For the down-stream face:

$$q = mp \qquad (28)$$

$$p' = m^2 p \qquad (29)$$

and,

$$f_1 = \frac{p}{\cos^2\theta} = p \sec^2\theta \qquad (30)$$

A check of Equation (30) at $x = L$, is as follows: In Equation (25) let $q = mp$; $p' = m^2p$, and $m = \tan\theta$; then,

$$f_1 = \tfrac{1}{2}\left(p + m^2p + \sqrt{(p - m^2p)^2 + 4\,m^2p^2}\right) = p + m^2p = p\,(1 + m^2) \quad (31)$$

but, $\tan^2\theta + 1 = m^2 + 1 = \sec^2\theta$, or $f_1 = p\,(1 + m^2) = p \sec^2\theta$, which checks.

(2) The total internal shear on any horizontal plane must be equal to the horizontal component of the total water pressure above that plane, or,

$$\int_0^L q\,dx = Q = H \qquad (32)$$

(3) The sum of the first and second principal stress is equal to the sum of the normal stresses at any point, or,

$$f_1 + f_2 = p + p' \qquad (33)$$

(4) The secondary principal stress, f_2, on the down-stream face is equal to zero.

For special cases:

(5) When the up-stream face is vertical, the shear is equal to zero at the up-stream face, or, when $n = 0$, $q_{(x=0)} = 0$.

(6) When the up-stream face is vertical, the horizontal normal stress at the up-stream face is equal to the unit water pressure, or when $n = 0$, $p'_{(x=0)} = F = \frac{h}{k}$.

(7) When the down-stream face is vertical, the shear is equal to zero at the down-stream face, or, when $m = 0$, $q_{(x=L)} = 0$.

(8) When the down-stream face is vertical, the horizontal normal stress at the down-stream face is equal to zero, or, when $m = 0$, $p'_{(x=L)} = 0$.

Stress Distribution in Gravity Dams Straight in Plan—

Example 1.—Given a triangular dam with water load to the top, with the up-stream face vertical, and with the resultant of forces falling at the middle-third point. Let $n = 0$; $m = \frac{1}{k^{\frac{1}{2}}}$; h = height; and $L = hm$ = base length.

The up-stream vertical normal stress is zero at any elevation, and $S = \frac{p_a - p_b}{L}$ is a constant $= \frac{h}{L}$, or $p = S\,x = \frac{x}{m} = k^{\frac{1}{2}}\,x$.

As the up-stream face is vertical, $n = 0$ and $V_1 = 0$, and from Equations (10), (11), and (12), $A_1 = 0$; $B_1 = 1$; and $C_1 = 0$.

Similarly, by substituting the values of $n = 0$ and $V_2 = 0$ in Equations (15), (16), and (17), $A_2 = 0$; $B_2 = 1$; and $C_2 = 0$. Substituting in Equation (19), $q = x$; and, in Equation (22), $p' = F + Q_1 - Q_2 = F + 0 - 0 = F$.

The three equations for the determination of stress are:

$$p = k^{\frac{1}{2}}\,x \quad \text{.........................} (34)$$

$$q = x \quad \text{.........................} (35)$$

and,

$$p' = F \quad \text{.........................} (36)$$

The shear stress for a triangular dam varies along a horizontal section as a straight line and the horizontal normal stress is a constant, but the principal stresses do not vary as a straight line along a horizontal section. The numerical computations for a dam 100 ft. high, with a 66-ft. base, meeting the requirements of Example 1 follows. For convenience, L is chosen as a whole number and as the resultant falls on the one-third point, the weight of the concrete is determined as 143.48 lb. per cu. ft.

The value of k is $\frac{143.48}{62.50} = 2.296$; $L = \frac{h}{\sqrt{k}} = 0.66\,h$; $W = \frac{0.66\,h^2}{2}$ $= 0.33\,h^2$; and, $H = \frac{62.50}{143.48} \times \frac{h^2}{2} = \frac{h^2}{2.296 \times 2} = 0.2178\,h^2$.

Taking moments about the down-stream toe (Fig. 4): $0.44h \times 0.33h^2 - 0.2178h^2 \times \frac{h}{3} = 0.1452h^3 - 0.0726h^3 = 0.0726h^3$.

The point of application of the forces $= \dfrac{0.0726h^3}{0.33h^2} = 0.22h$, or, at the one-third point. By Equation (1),

$$p = \frac{P}{a} \pm \frac{6\,M}{L^2} = \frac{0.33\,h^2}{0.66\,h} + \frac{6 \times 0.11\,h \times 0.33\,h^2}{(0.66\,h)^2} = 0.5\,h \pm \frac{0.2178\,h}{0.4356} = 0.5h \pm 0.5h$$

In this example, $p_b = 0$, and $p_a = h$; therefore, $S = \dfrac{h}{L} = \dfrac{h}{0.66\,h} = 1.51515$ = constant.

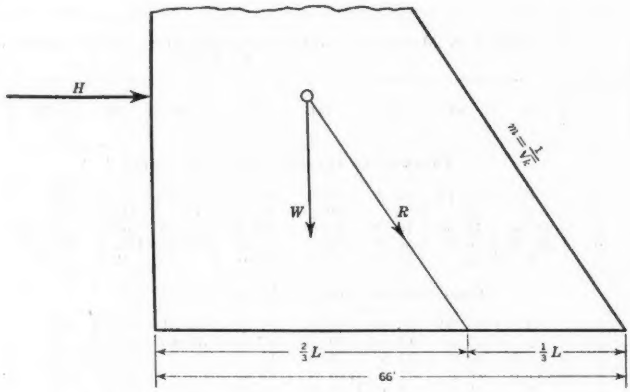

FIG. 4.

For the same distance, x, on any plane, $P_u = P_o = P_d$, and, by Equations (34), (35), and (36).

$$p = 1.51515\,x \quad \text{.......................}(37)$$

$$q = P_u + x - P_o = x \quad \text{.......................}(38)$$

and,

$$p' = F + Q_1 - Q_2 = F = 0.4356\,h \quad \text{...............}(39)$$

For a 100-ft. dam, $h = 100$; $L = 66$ ft., and,

$$p = 1.5151\,x \quad \text{.......................}(40)$$

$$q = x \quad \text{.......................}(41)$$

$$p' = 43.56 \quad \text{.......................}(42)$$

Substituting $x = 0$ and $x = L$ for the up-stream and down-stream faces, respectively, in Equations (40), (41), and (42), $p' = 43.56$ at both faces; at the up-stream face, both p and q are equal to zero; and at the down-stream face, $p = 100$ and $q = 66$.

Applying the eight itemized checks to Equations (40), (41), and (42), for the up-stream face, $q = 0$ by Equation (26) (Table 1, Item No. 2), and $p' = F - qn$; for the down-stream face, $q = 66$ by Equation (28), $p' = 43.56$

by Equation (29) (Table 1, Item No. 3); and $f_1 = 143.56$ by Equation (30) (Table 1, Item No. 4). By Equation (32), $Q = H = \frac{66^2}{2} = 2\,178$; but $H = \frac{h^2}{2k} = \frac{10\,000}{4.59136} = 2\,178$, which checks. Equation (33) is checked at all values of x in Table 1 (Items Nos. 1 to 5).

TABLE 1.—Stresses in Various Types of Gravity Dams

Item No.	Factors	Stress in Masonry Units per Square Foot, for Various Values of x, in Feet									
		0	10	20	30	40	50	60	66	70	80
		Triangular Gravity Dam — Example 1									
1........	p	0	15.15	30.3	45.4	60.6	75.76	90.9	100	.....	......
2........	q	0	10	20	30	40	50	60	66	.....	......
3........	p'	43.56	43.56	43.56	43.56	43.56	43.56	43.56	43.56	.....	......
4........	f_1	43.56	46.7	58.0	74.5	93.6	112.16	131.7	143.56	.....	......
5........	f_2	0	12	15.8	14.5	10.6	7.16	2.7	0	.....	......
		Rectangular Gravity Dam — Example 2									
6........	p	0	30.30	60.60	90.91	121.21	151.51	181.81	200	.....	......
7........	q	0	25.46	41.84	49.12	47.31	36.40	16.41	0	.....	......
8........	p'	43.56	40.9	34.0	24.8	15.1	6.7	1.4	0	.....	......
9........	f_1	43.56	67.2	91.2	116.8	139.0	160.1	183.1	200	.....	......
10........	f_2	0	4.0	3.42	—1.15	—2.8	—1.9	+0.1	0	.....	......
		Gravity Dam, Arched in Plan — Example 4									
11........	p	33.62	37.88	42.14	46.40	50.66	54.92	59.18	.....	63.44	67.70
12........	q	0	6.75	13.52	20.30	27.1	33.9	40.7	.....	47.4	54.156
13........	p'	41.67	42.	42.2	42.6	42.6	43.0	42.8	.....	43.2	43.325
14........	f_1	41.67	47.01	55.7	64.9	74.1	83.4	92.5	.....	101.8	111.0
15........	f_2	33.62	32.87	28.7	24.1	19.1	14.5	9.5	.....	8.4	0

The secondary principal stress, f_2 at the down-stream face is equal to zero (see Table 1, Item No. 5), and the stress at the up-stream face is equal to zero, in accordance with Checks (4) and (5). Finally, the horizontal normal stress at the up-stream face is equal to the unit water pressure, which is in accordance with Check (6).

A common method showing the distribution of the first and second principal stresses is to draw lines of equal stress or contours of principal stress, as illustrated in Fig. 5 for Example 1. The method of showing the stress distribution by means of the ellipse of stress (Fig. 6) was introduced by Messrs. J. S. Wilson and William Gore.[6] The major and minor axes indicate both magnitude and direction of the principal stresses.

Example 2.—Assume a restangular dam with the up-stream face vertical and with the resultant of forces falling at the middle-third point at a height, $h, = 100$ ft. Let $L = 66$ ft., and let n, m, and $V = 0$ (see Fig. 7). Then, $P = 66\ h$, or 6 600 masonry units, and, $H = \frac{62.5}{143.48} \times \frac{h^2}{2} = 0.2178\ h^2$.

[6] *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Pt. II, 1908.

Taking moments about the down-stream toe, $M_L = 145\,200$. The point of application of the resultant of forces $= \dfrac{145\,200}{6\,600} = 22$ ft., which is the third point. By Equation (1), $p_o = 0$; $p_a = 200$; and $S_o = 3.0303$. For the sec-

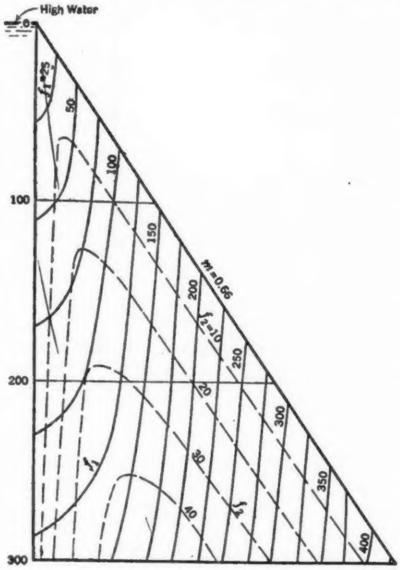

FIG. 5.

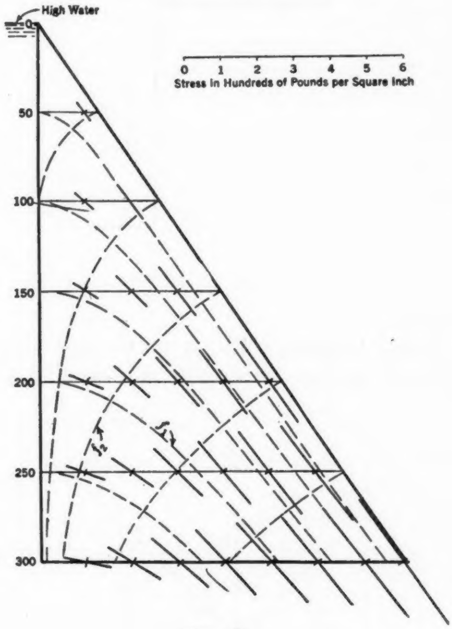

FIG. 6.

tion 1 ft. above the base: $P_u = 66 \times 99 = 6\,534$. The moment about the down-stream toe equals 145 198. The point of application of the resultant of forces is 22.22 ft. from the down-stream face, or 10.78 ft. from the center of the base. Equation (1) gives, $p = \dfrac{6\,534}{66} \pm \dfrac{6\,534 \times 10.78}{66} \times \dfrac{6}{66} = 99 \pm 97.030$; that is, $p_u = 1.97$; $p_a = 196.03$; and $S_u = 2.940$.

Similarly, for the section 1 ft. below the base, $P_d = 6\,666$; the moment about the toe is 145 178; the point of application of the resultant of forces is 21.78 ft. from the down-stream face; and, $p = 101 \pm 103.030$; that is, $p_d = -2.03$; $p_a = 204.03$; and $S_b = 3.12$.

Solving Equations (10), (11), and (12), for these conditions, $A_1 = 0$; $B_1 = 2.97$; and $C_1 = 0.045$. Similarly, for Equations (15), (16), and (17), $A_2 = 0$; $B_2 = 3.030$; and $C_2 = 0.459$. It follows from Equation (3) that,

$$p = 3.0303\,x \qquad (43)$$

With the foregoing values determined, make the proper substitutions in Equations (19) and (23); thus,

$$q = 3\,x - 0.045\,x^2 \qquad (44)$$

and,

$$p' = 43.56 - 0.03\,x^2 + 0.0003\,x^3 \qquad (45)$$

The first and second principal stresses are determined by substituting Equations (43), (44), and (45) in Equation (25). The results are listed in

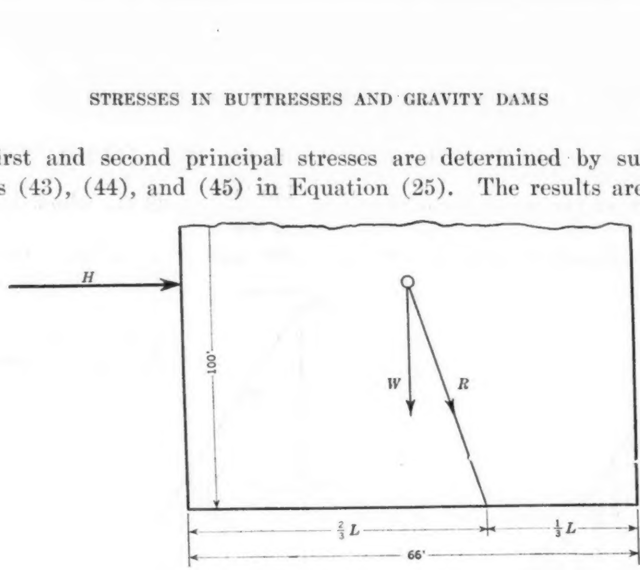

FIG. 7.

Table 1 (Items Nos. 6 to 10, inclusive). It will be noted that there is a small amount of tension in a dam of this type.

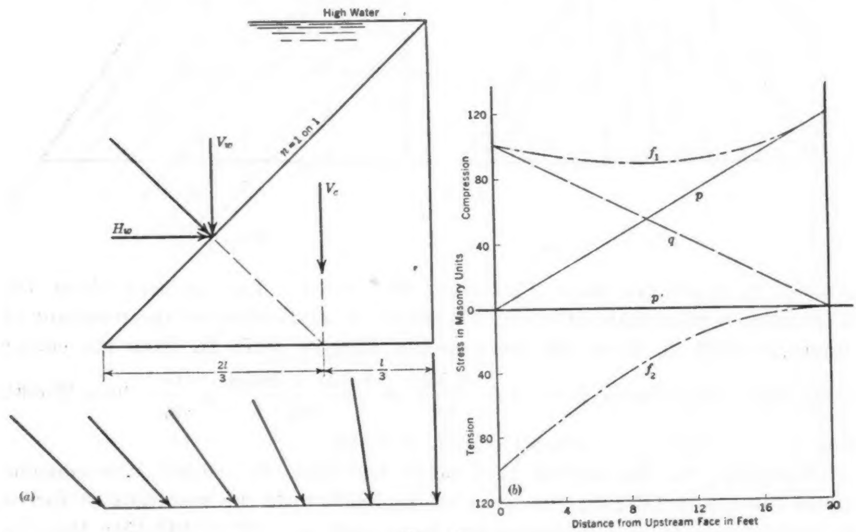

FIG. 8.

When plotted graphically the shear distribution is parabolical in shape. As in Example 1, the total shear, determined by Equation (32), is $Q = 2\,178$. The average shear is $\frac{2\,178}{66} = 33$, and the maximum vertical shear is $1.5 \times 33 = 49.5$. The results are checked, item for item, as demonstrated in Example 1.

Example 3.—Assume a triangular buttress type dam with the up-stream face on a 1 to 1 slope, or $n = 1$; the down-stream slope is vertical, or $m = 0$;

the buttress spacing is twelve times the buttress thickness; and, for simplicity, the density of the deck slab is the same as the density of water, and the weight of 1 cu. ft. of masonry is assumed as 150 lb. (See Fig. 8(*a*)).

The three equations for the determination of stresses are:

$$p = 6\,x \quad \text{(46a)}$$

$$q = 5\,h - 5\,x \quad \text{(46b)}$$

and,

$$p' = 0 \quad \text{(46c)}$$

A graphical presentation of the principal stresses is shown in Fig. 8(*b*) for a dam 20 ft. high. Tensions would become excessive if this design were carried to any reasonable height.

Group II.—Gravity Dams Arched in Plan

Vertical Normal Stress.—The vertical normal stress on the up-stream edge of the trapezoidal section computed by the usual formula is:

$$\frac{P}{a} - \frac{M c_2}{I} = p_a \quad \text{(47)}$$

and the vertical normal stress on the down-stream edge of the trapezoidal

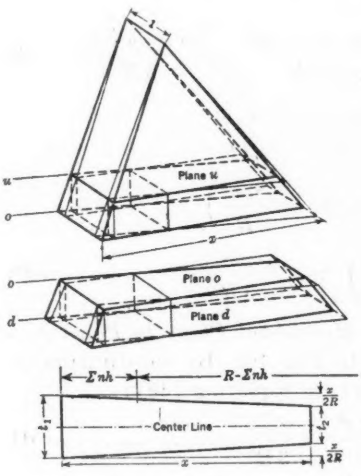

Fig. 9.

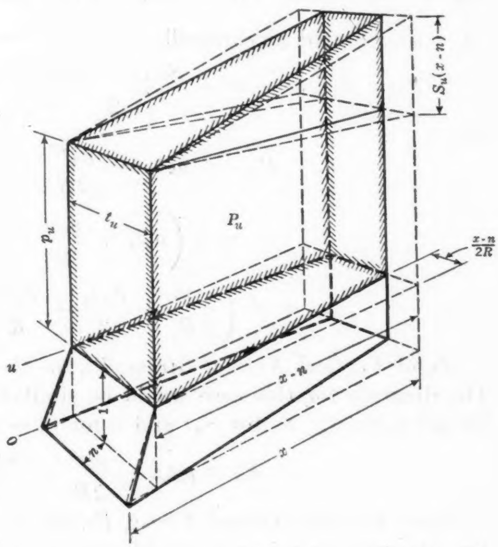

Fig. 10.

section (Fig. 9), is:

$$\frac{P}{a} + \frac{M c_1}{I} = p_b \quad \text{(48)}$$

in which, P is the sum of the vertical normal stresses on the horizontal plane; a, the area of the section; M, the external moment about the neutral

line; t_1, the thickness of section at the up-stream face; t_2, the thickness of section at the down-stream face; $c_1 = \frac{L}{3}\left(\frac{t_1 + 2t_2}{t_1 + t_2}\right)$, the distance from the neutral line to the down-stream face; $c_2 = \frac{L}{3}\left(\frac{t_2 + 2t_1}{t_1 + t_2}\right)$, the distance from the neutral line to the up-stream face; and I, moment of inertia of the sections about the neutral line; that is,

$$I = L^3 \frac{(t_1^2 + 4t_1t_2 + t_2^2)}{36\,(t_1 + t_2)}$$

Since t_2 will be written in terms of t_1 and x it does not appear in the formulas, and t will be used for the width at the up-stream face omitting the subscript, 1; but the subscripts u, o, and d, will be used to denote the plane on which t_1 is measured.

Total Vertical Normal Stress, P_u, on the Horizontal Plane to Point x.— The volume of the solid, P_u (shown in shaded lines in Fig. 10) is equal to the volume of the parallelopiped, p_u, t_u, $(x - n)$, plus the volume of the wedge, t_u, $(x - n)$, $S_u\,(x - n)$, minus the volume of the two wedges, $\frac{x - n}{2R}$, p_u, $(x - n)$, minus the volume of the two pyramids, $\frac{x - n}{2R}$, $S_u\,(x - n)$, $(x - n)$. Stated algebraically,

$$P_u = p_u t_u (x - n) + \frac{S_u (x - n)^2 t_u}{2} - \frac{p_u (x - n)^2}{2R} - \frac{(x - n)^3 S_u}{3R} \quad \ldots (49)$$

or,

$$P_u = -p_u t_u n - \frac{p_u n^2}{2R} + \frac{S_u t_u n^2}{2} + \frac{S_u n^3}{3R}$$
$$+ x\left(p_u t_u + \frac{p_u n}{R} - S_u n t_u - \frac{S_u n^2}{R}\right)$$
$$+ x^2\left(\frac{-p_u}{2R} + \frac{S_u t_u}{2} + \frac{S_u n}{R}\right) + x^3\,\frac{(-S_u)}{3R} \quad \ldots\ldots\ldots\ldots (50)$$

Total Vertical Normal Stress, P_o, on the Horizontal Plane to Point x.— The diagram for this case would be similar to Fig. 10. By substituting p_o for p_u; t_o for t_u; S_o for S_u; and x for $(x - n)$, in Equation (49):

$$P_o = p_o t_o x - \frac{p_o x^2}{2R} + \frac{S_o t_o x^2}{2} - \frac{S_o x^3}{3R} \quad \ldots\ldots\ldots\ldots\ldots (51)$$

Total Vertical Normal Stress, P_d, on the Horizontal Plane to Point x.— The diagram for this case would also be similar to Fig. 10. By substituting p_d for p_u; t_d for t_u; S_d for S_u; and $(x + n)$ for $(x - n)$ in Equation (49):

$$P_d = p_d t_d n - \frac{p_d n^2}{2R} + \frac{S_d n^2 t_d}{2} - \frac{S_d n^3}{3R} + x\left(p_d t_d - \frac{p_d n}{R} + S_d t_d n - \frac{S_d n^2}{R}\right)$$
$$+ x^2\left(\frac{-p_d}{2R} + \frac{S_d t_d}{2} - \frac{S_d n}{R}\right) + x^3\left(\frac{-S_d}{3R}\right) \quad \ldots\ldots\ldots (52)$$

Volume W_1, of the Section Between Planes u and o.—The volume of W_1 (shown in solid line in Fig. 11) is equal to the volume of the parallelopiped, t_o, x, 1, minus the two wedges, t_o, n; minus the two wedges, $\frac{x}{2R}$, x; plus the two inverted pyramids, n, $\frac{n}{2R}$, 1, which has been subtracted with the two wedges, t_o, n. Stated algebraically,

$$W_1 = t_o\,x - \frac{x^2}{2R} - \frac{t_o\,n}{2} + \frac{n^2}{6R} \qquad (53)$$

Volume, W_2, of the Section Between Planes o and d.—The volume of W_2 (shown in solid lines of Fig. 12) equals,

$$W_2 = t_o\,x - \frac{x^2}{2R} + \frac{t_o\,n}{2} + \frac{n^2}{6R} \qquad (54)$$

Shearing Stresses.—The vertical shear, q_1, at any point, x, on the section between Planes u and o is obtained from the basic assumption expressed by Equation (8).

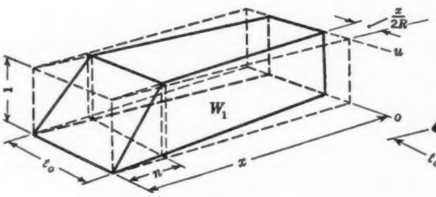

FIG. 11.

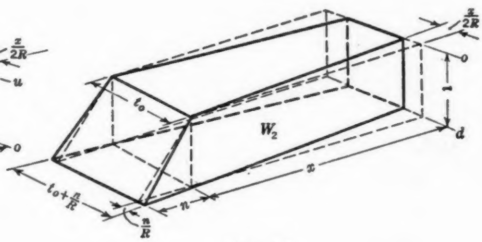

FIG. 12.

Substituting the values of P_u, P_o, and W_1 from Equations (50), (51), and (53), Equation (8) becomes,

$$\begin{aligned} q_1 = {} & V_1 - \frac{n\,t_o}{2} - p_u\,t_u\,n - \frac{p_u\,n^2}{2} + \frac{S_u\,t_u\,n^2}{2} + \frac{S_u\,n^3}{3R} + \frac{n^2}{6R} \\ & + x\left(p_u\,t_u + t_o - p_o\,t_o + \frac{p_u\,n}{R} - S_u\,nt_u - \frac{S_u\,n^2}{R}\right) \\ & + x^2\left(\frac{-p_u}{2R} + \frac{S_u\,t_u}{2} - \frac{1}{2R} + \frac{p_o}{2R} - \frac{S_o\,t_o}{2} + \frac{S_u\,n}{R}\right) + x^3\left(-\frac{S_u}{3R} + \frac{S_o}{3R}\right) \end{aligned} \qquad (55)$$

Let,

$$A_1 = V_1 - \frac{n\,t_o}{2} - p_u\,t_u\,n - \frac{p_u\,n^2}{2R} + \frac{S_u\,t_u\,n^2}{2} + \frac{S_u\,n^3}{3R} + \frac{n^2}{6R} \qquad (56)$$

$$B_1 = p_u\,t_u + t_o - p_o\,t_o + \frac{p_u\,n}{R} - S_u\,nt_u - \frac{S_u\,n^2}{R} \qquad (57)$$

$$C_1 = \frac{-p_u}{2R} + \frac{S_u\,t_u}{2} - \frac{1}{2R} + \frac{p_o}{2R} + \frac{S_o\,t_o}{2} + \frac{S_u\,n}{R} \qquad (58)$$

and,

$$D_1 = \frac{S_u + S_o}{3R} \qquad (59)$$

Then,

$$q_1 = A_1 + B_1 x + C_1 x^2 + D_1 x^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (60)$$

Shear in the section between Planes o and d is obtained from $\Sigma V = 0$, or,

$$q_2 = P_o + V_2 + W_2 - P_d = V_2 + \frac{n t_o}{2} - p_d t_d n + \frac{p_d n^2}{2R} - \frac{S_d n^2 t_d}{2} + \frac{S_d n^3}{3R}$$

$$+ \frac{n^2}{6R} + x \left(p_o t_o + t_o - p_d t_d + \frac{p_d n}{R} - S_d t_d n + \frac{S_d n^2}{R} \right)$$

$$+ x^2 \left(\frac{-p_o}{2R} + \frac{S_o t_o}{2} - \frac{1}{2R} + \frac{p_d}{2R} - \frac{S_d t_d}{2} + \frac{S_d n}{R} \right) + x^3 \left(\frac{-S_o + S_d}{3R} \right) \ldots (61)$$

Let,

$$A_2 = V_2 + \frac{n t_o}{2} - p_d t_d n + \frac{p_d n^2}{2R} - \frac{S_d n^2 t_d}{2} + \frac{S_d n^3}{3R} + \frac{n^2}{6R} \quad \ldots\ldots (62)$$

$$B_2 = p_o t_o + t_o - p_d t_d + \frac{p_d n}{R} - S_d t_d n + \frac{S_d n^2}{R} \quad \ldots\ldots\ldots (63)$$

$$C_2 = \frac{-p_o}{2R} + \frac{S_o t_o}{2} - \frac{1}{2R} + \frac{p_d}{2R} - \frac{S_d t_d}{2} + \frac{S_d n}{R} \quad \ldots\ldots\ldots (64)$$

and,

$$D_2 = \frac{-S_o + S_d}{3R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (65)$$

Then,

$$q_2 = A_2 + B_2 x + C_2 x^2 + D_2 x^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (66)$$

The average of q_1 and q_2 is the average shear on the vertical plane at a distance, x, from the up-stream face, or,

$$q_t = \frac{q_1 + q_2}{2} = \frac{A_1 + A_2}{2} + \frac{(B_1 + B_2)\, x}{2} + \frac{(C_1 + C_2)\, x^2}{2} + \frac{(D_1 + D_2)\, x^3}{2} \ldots (67)$$

and for horizontal normal stresses,

$$p'_t = F + Q_1 - Q_2 = F + \int_{0 \cdot 5 n}^{x} q_1 \, dx - \int_{0 \cdot 5 n}^{x} q_2 \, dx = F - \frac{n}{2} (A_1 + A_2)$$

$$+ \frac{n^2}{8} (B_2 - B_1) - \frac{n^3}{24} (C_1 + C_2) + \frac{n^4}{64} (D_2 - D_1) + x (A_1 - A_2) + x^2 \frac{(B_1 - B_2)}{2}$$

$$+ x^3 \frac{(C_1 - C_2)}{3} + x^4 \frac{(D_1 - D_2)}{4} \quad \ldots\ldots\ldots\ldots\ldots\ldots (68)$$

The stresses, q_t and p'_t in Equations (67) and (68) are not unit stresses at a point, but for the total width of a section. To obtain unit stress the

values must be divided by the width of the section at that point. When $R = \infty$; $\frac{1}{R} = 0$, and $t_u = t_d = 1$; Equations (56), (57), (58), (59), (62), (63), (64), and (65) reduce to Equations (10), (11), (12), (15), (16), and (17), respectively.

First and Second Principal Stresses.—The general equation (Equation (25)) for the first and second principal stresses for the case of a straight gravity dam, also apply for this case of gravity dams arched in plan.

Checks of Equations.—All checks for shearing stress, horizontal normal stress, and principal stresses at up-stream and down-stream faces of straight gravity dams apply for this case of gravity dams arched in plan.

Stress Distribution in Gravity Dams Arched in Plan—

Example 4.—Consider a triangular dam with up-stream face vertical, or $n = 0$. Other dimensions are: $m = 0.8$; $R = 1\,400$ ft.; height of dam, $h = 100$ ft.; the weight of 1 cu. ft. of masonry = 150 lb.; and the weight of 1 cu. ft. of water = 62.5 lb. Values for p and S are obtained by the usual rules of geometry and mechanic. The results are summarized in Table 2.

TABLE 2.—Unit Normal Pressures, Gravity Dam Arched in Plan

h	p_b	p_a	S
99u	33.2957	66.9918	0.42545
100o	33.6189	67.6955	0.42595
101d	33.9418	68.3998	0.42646

Substituting values of Table 2, together with values of R, n, and t, the values of the factors, A, B, C, and D, in Equations (56), (57), (58), (59), (62), (63), (64), and (65) are obtained. Using these factors in Equations (3), (67), and (68), there follows: $p = 33.6181 + 0.42596\,x$; $q_t = 0.67696\,x - 0.0004932\,x^2 + 0.0000001197\,x^3$; and $p'_t = 41.666 - 0.0001362\,x^2 - 0.00000010579\,x^3 - 0.0000000000126\,x^4$.

In Table 1, Items Nos. 11 to 15, inclusive, are the stresses in a gravity dam, arched in plan, at a height of 100 ft. The radius is 1 400 ft., the up-stream face is vertical, and the down-stream slope has a batter of 1 on 0.8.

Group III.—Tapering Buttresses

The formulas developed for the stress distribution in a gravity dam either straight or arched in plan are not directly applicable for the stress distribution in the buttress of slab and buttress or arch and buttress types of dams that taper in thickness with the height. The taper of the buttress must be included in any formula for the stress analysis. The assumption is made that the slab or arch loading and the normal component of slab weight are applied normal to the face of the buttress, but the weight of the arch is applied vertically to the buttress. These assumptions enable the distribution of the vertical normal stress on the buttress to be written algebraically. With a reduction of the vertical normal stress near the up-stream

face because of distribution over the greater area of deck and buttress, the shearing stress would be greater. Any increase in shear or decrease in the vertical normal stress, or both, decreases the second principal stress (increases the tension). Fred A. Noetzli, M. Am. Soc. C. E., has discussed this point in detail.[7]

Vertical Normal Stress.—The vertical normal stress on any rectangular plane computed by the usual formula is,

$$\frac{1}{t}\left(\frac{P}{L} \pm \frac{6\,M}{L^2}\right) = p_a \text{ or } p_b \quad \text{.................. (69)}$$

For a tapering buttress, t must vary with the height. For convenience, the section generally analyzed is taken as a unit width and in this case the unit section is taken on Plane o. (See Fig. 12.)

Let t_u, t, and t_d be the total thickness as of buttress at upper, middle, and lower planes, respectively. The thickness of the upper section compared to that of the center section is in the ratio of $\frac{t_u}{t} = b_u$.

A stress, p, on Plane u, when distributed on Plane o is carried by a greater area; hence, the stress on Plane u may be considered as a smaller stress acting on a unit area on Plane o; thus, on Plane u:

$$p = p_u + S_u\, x_u \quad \text{...................... (70)}$$

To apply p of Plane u on Plane o, Equation (70) must be multiplied by the ratio of the thickness of Planes u and o, or,

$$p = p_u\, b_u + S_u\, b_u\, x_u \quad \text{................... (71)}$$

and the total vertical normal stress on Plane u is,

$$P_u = p_u\, b_u\, x_u + \frac{S_u\, b_u\, x_u^2}{2} \quad \text{................ (72)}$$

The thickness of the lower section compared to that of the center section is in the ratio of $\frac{t_d}{t}$. Let $\frac{t_d}{t} = b_d$; hence, Equation (73) is similar to Equation (72):

$$P_d = p_d\, b_d\, x_d + \frac{S_d\, b_d\, x_d^2}{2} \quad \text{.................... (73)}$$

The total vertical normal stress on the center plane is,

$$P_o = p_o\, x + \frac{S_o\, x^2}{2} \quad \text{...................... (74)}$$

Let W_1 and W_2 be the weight in masonry units of the unit width of concrete at Plane o between Planes u and o and between Planes o and d,

[7] "Design and Construction of Masonry Dams," by Edward Wegmann, M. Am. Soc. C. E., Eighth Edition.

respectively. From an inspection of Fig. 13 it is seen that,

$$W_1 = \frac{(1 + b_u)\,x}{2} - \frac{b_u\,n}{3} - \frac{n}{6} \quad \text{.................(75)}$$

and,

$$W_2 = \frac{(1 + b_d)\,x}{2} + \frac{b_d\,n}{3} + \frac{n}{6} \quad \text{..............(76)}$$

Shearing Stresses.—The vertical shear, q_1, at the point, x, on the section between Planes u and o is obtained from the assumption that the summation

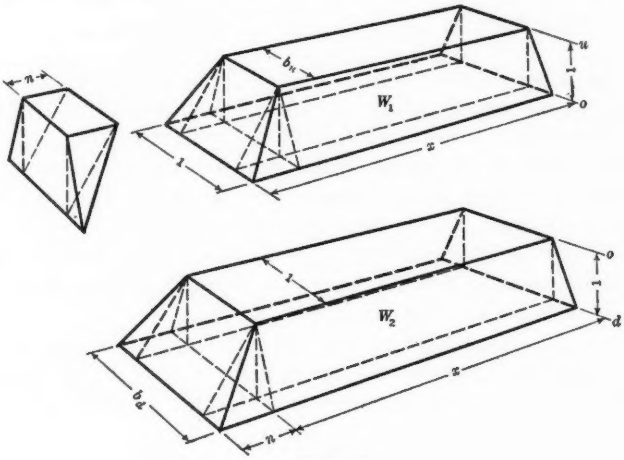

FIG. 13.

of the vertical forces is equal to zero, or, $\Sigma V = 0$. In Equation (8) substitute values of P_u, P_o, and W_1, from Equations (72), (6), and (75), and for x_u, substitute $x - n$; then, combining terms,

$$q_1 = \left(V_1 - \frac{n}{6} - \frac{b_u\,n}{3} - p_u\,b_u\,n + \frac{S_u\,b_u\,n^2}{2}\right)$$

$$+\; x\left(-p_o + \tfrac{1}{2} + \frac{b_u}{2} + p_u\,b_u - S_u\,b_u\,n\right) + x^2\left(\frac{S_u\,b_u}{2} - \frac{S_o}{2}\right) \quad \text{..(77)}$$

Let,

$$A_1 = V_1 - \frac{n}{6} - \frac{b_u\,n}{3} - p_u\,b_u\,n + \frac{S_u\,b_u\,n^2}{2} \quad \text{.........(78)}$$

$$B_1 = -p_o + \tfrac{1}{2} + \frac{b_u}{2} + p_u\,b_u - S_u\,b_u\,n \quad \text{................(79)}$$

$$C_1 = \frac{S_u\,b_u}{2} - \frac{S_o}{2} \quad \text{..................................(80)}$$

Then, Equation (80) may be reduced to the form of Equation (13), or,

$$q_1 = A_1 + B_1\,x + C_1\,x^2 \quad \text{....................(81)}$$

Likewise, by a similar process, q_2 reduces to the form of Equation (18) and,

$$A_2 = V_2 + \frac{n}{6} + \frac{b_d\,n}{3} - p_d\,b_d\,n - \frac{S_d\,b_d\,n^2}{2} \qquad (82)$$

$$B_2 = p_o + \tfrac{1}{2} + \frac{b_d}{2} - p_d\,b_d - S_d\,b_d\,n^2 \qquad (83)$$

$$C_2 = \frac{S_o + S_d\,b_d}{2} \qquad (84)$$

or,

$$q_2 = A_2 + B_2\,x + C_2\,x^2 \qquad (85)$$

When the taper equals zero, b is equal to 1, Equations (78), (79), and (80) for q_1 and its corresponding form for q_2, Equations (82), (83), and (84), reduce to Equations (10), (11), (12), (15), (16), and (17).

Horizontal Normal Stresses.—The horizontal normal stress, at any point, x, is obtained by the use of the assumption that the summation of the horizontal stresses is equal to zero (see derivation of Equation (23)).

First and Second Principal Stresses.—The general equation for first and second principal stresses for the case of a straight gravity dam also applies for this case of tapering buttresses.

Checks of Equations.—All checks for shearing stress, horizontal normal stress, and principal stresses at the up-stream and down-stream faces of straight gravity dams apply for this case of tapering buttresses.

Conclusion

Middle-Third Theory.—The numerical examples indicate the fallacy of the middle-third theory as a safe criterion for dam design.

In rectangular gravity dams the volume of material is twice that in triangular dams and the up-stream half of the dam has very limited resistance to uplift, while the down-stream half has a tension in the second principal stress making the former a less stable structure than the latter (compare Items Nos. 5 and 10, Table 1.)

Many unreinforced buttresses of the type shown in Fig. 8(*a*) are standing and are considered safe, but high tensions exist in the second principal stress, and provisions should be made for this stress in the design of this type. By making only a slight change in the quantity of material in the buttress (changing the down-stream slope), considerable tension is eliminated.

New Design—Buttress and Gravity Dams.—In the design of dams of the buttress type the tension at the up-stream face should be held at a minimum. From an inspection of Mohr's diagrams it is evident that the tension is reduced if the shear, q, is reduced. The vertical shear at the up-stream face is,

$$q_y = n\left(p - \frac{V}{t}\right) \qquad (86)$$

in which, t is the thickness of the buttress; V, the unit water pressure plus

the vertical component of the weight of the deck on a 1-ft. section at any elevation. An increase of the up-stream slope decreases n, the tangent of slope measured from the vertical, and, hence, decreases the shear, and if the up-stream slope were vertical ($n = 0$) the shear would be zero. However, as the slope decreases the resisting moment, due to the vertical component of the weight of water, is decreased, and the ratio of horizontal forces to vertical forces (sliding factor) is increased. Fredrik Vogt, Assoc. M. Am. Soc. C. E., has shown[8] that the most economical design is that in which $n = 0.5\pm$. (The average slope for most American multiple-arch or slab-and-buttress dams is about 1.) The use of these slopes, while giving good stress distribution results, involves a sliding factor in excess of that generally accepted by engineers as adequate for some types of foundation footings. It is probable that the most economical design for buttress dams will have an up-stream slope between 1 on $\frac{1}{2}$ and 1 on 1 and one or more contraction joints along the line of principal stress.

The writer made an effort to determine the stresses in the San Gabriel Dam with inclined contraction joints, but was unable to decide upon any satisfactory assumption of loading or pressures that might exist on the contraction joints as designed. It is his opinion, however, that the derived formulas will indicate a system of joints located so that satisfactory assumptions for determining a design can be made.

Distribution of Stresses.—It is well known that the vertical normal stress distribution of gravity dams is not linear, although for triangular dams of thicknesses of 50 to 60 ft., the error probably is small. When and if a more exact distribution of the vertical normal stress is expressed in terms of the base length, it should be inserted in the foregoing formulas and the shear and horizontal normal stresses will not be distributed linearly. Some large-scale experimental work on stress distribution in mass concrete would be valuable to the profession, although a more intimate knowledge of other physical properties of concrete is more essential.

By the use of methods now in practice the principal stresses at the heel and toe of small gravity or buttress dams, either arched or straight in plan, are easily obtained after finding the vertical normal stresses. The shear and horizontal normal stresses may be assumed as being distributed linearly for only a monolithical triangular dam with water to the apex of the triangle (see Table 1 and Fig. 8). The shear and horizontal normal stresses for rectangular and irregular sections are not distributed linearly (see Table 1) and an analytical method of analysis is necessary to determine their magnitude and direction.

The gravity dam as designed must transmit shear from one portion of the section to another portion. If the section is jointed or ruptured from any cause, when the load is applied, shear can not be transferred completely across the line of rupture. As temperature and shrinkage stresses cause partial or total failure, dividing the base into irregular sections, the section modulus of the entire base should not be used in Equation (1). Any gravity

[8] "Economical Design of Buttresses for High Dams and of Cellular Gravity Dams."

dam higher than, say, 100 ft., cannot be made monolithically; hence, each monolithic block into which the dam is divided by cracks or joints (which may not be triangular) should be analyzed.

The analytical method described in this paper, when used for gravity or buttress dams, has at least five advantages:

(1) It provides an exact and concise method of determining maximum stresses;
(2) It gives complete internal stress analysis;
(3) It indicates that the monolithic section only should be used when determining the section modulus of the base;
(4) It suggests a method of design of buttresses eliminating tension; and,
(5) It makes available a basis for design of construction joints.

APPENDIX I

Notation

The symbols used in this paper are as follows:

a = area of section analyzed.

b = a ratio of thicknesses, t; that is, $b_u = \frac{t_u}{t_o}$.

c = the distance of the neutral line from the faces: c_1, from the down-stream face; c_2, from the up-stream face.

d = the horizontal plane 1 ft. below Plane o.

f = unit stress: f_1 = first principal stress; and f_2 = second principal stress.

h = height.

k = ratio of the weights of a unit of masonry to a unit of water.

m = batter at the down-stream face = horizontal distance: 1 unit vertical.

n = batter at the up-stream face = horizontal distance: 1 unit vertical.

o = the horizontal plane through the section considered.

p = unit vertical normal pressure at any point, x, on a plane; p_b = vertical pressure at up-stream face; p_a = vertical pressure at down-stream face.

p' = unit horizontal normal pressure at any point, x, on a plane.

q = unit vertical or horizontal shear at any point on a plane; q_1 = shear between Planes u and o; and q_2 = shear between Planes o and d.

t = thickness of the section analyzed; in a straight gravity section, $t = 1$, and in a tapering buttress section, t = the total thickness at the middle plane; t_1 = thickness at up-stream face; t_2 = thickness at down-stream face.

u = a horizontal plane, 1 ft. above Plane o.

w = weight of one cubic unit of masonry.

x = horizontal distance from an origin of ordinates to the point to be analyzed; x_o = distance from the up-stream face of the section cut by Plane o; x_u = distance from the up-stream face of the section cut by Plane o; x_u = distance from the up-stream face of the section cut by Plane u; x_d = distance from the up-stream section cut by Plane d; etc.

$A =$ a substitution factor, with appropriate subscripts.
$B =$ a substitution factor, with appropriate subscripts.
$C =$ a substitution factor, with appropriate subscripts.
$D =$ a substitution factor, with appropriate subscripts.
$F =$ horizontal component of external forces applied on a section 1 ft. thick midway between Planes u and d.
$H =$ horizontal component of the resultant of all external forces applied to the structure above any plane.
$I =$ moment of inertia.
$L =$ the length of any horizontal section.
$M =$ bending moment.
$P =$ the sum of vertical normal stresses on any horizontal plane under consideration; P_u refers to the summation of stresses on Plane u, etc.
$Q =$ horizontal shear: Q_1 on the plane midway between Planes u and o; Q_2 on the plane midway between Planes o and d.
$R =$ radius of dam.
$S =$ the rate of change of intensity of the vertical normal stress; S_u refers to any point on Plane u, etc.
$V =$ the vertical component of the water or deck pressure on the up-stream face of a section between two limiting planes; V_1 is between Planes u and o; V_2 is between Planes o and d.
$W =$ weight of concrete included between two horizontal planes; W_1 is between Planes u and o; W_2 is between Planes o and d.
$\theta =$ direction of first principal stress measured from the vertical.

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DISCUSSIONS

ANALYSIS OF CONTINUOUS FRAMES BY DISTRIBUTING FIXED-END MOMENTS

Discussion

BY ALFRED GORDON, ASSOC. M. AM. SOC. C. E.

ALFRED GORDON,[31] ASSOC. M. AM. SOC. C. E. (by letter).[31a]—The method of analyzing rigid frames, described in this paper, is one of those maddeningly simple solutions, of a baffling problem, that show sheer genius. By a flight of imagination amounting to pure poetry, Professor Cross has made the analysis of indeterminate structures even simpler than that of a simple truss. The flood of discussion his paper has unloosed is proof of its extraordinary verbal economy, nothing having arisen that was not indicated therein.

Although it may be only "dotting the i's and crossing the t's," the writer would point out more exactly the relation between Professor Cross's method and the tables[32] compiled by Walter Ruppel, Assoc. M. Am. Soc. C. E. The slope-deflection equations for any beam subjected to moments at each end may be represented (using Mr. Ruppel's nomenclature), as follows:

$$M_{AB} = \frac{1 - v}{pl\,[1 - (u + v)]}\,\theta_A + \frac{v}{pl\,[1 - (u + v)]}\,\theta_B \qquad (90)$$

and,

$$M_{BA} = \frac{1 - u}{ql\,[1 - (u + v)]}\,\theta_B + \frac{u}{ql\,[1 - (u + v)]}\,\theta_A \qquad (91)$$

or,

$$M_{AB} = \alpha\,\theta_A + \beta\,\theta_B \qquad (92)$$

and,

$$M_{BA} = \gamma\,\theta_B + \delta\,\theta_A \qquad (93)$$

NOTE.—The paper by Hardy Cross, M. Am. Soc. C. E., was published in May, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1930, by Messrs. C. P. Vetter, L. E. Grinter, S. S. Gorman, A. A. Eremin and E. F. Bruhn; October, 1930, by Messrs. A. H. Finlay, R. F. Lyman, Jr., R. A. Caughey, Orrin H. Pilkey, and I. Oesterblom; November, 1930, by Messrs. Edward J. Bednarski, S. N. Mitra, Robert A. Black, and H. E. Wessman; January, 1931, by Messrs. Jens Egede Nielsen, F. E. Richart, and William A. Oliver; February, 1931, by Messrs. R. R. Martel and Clyde T. Morris; March, 1931, by Francis P. Witmer, M. Am. Soc. C. E.; May, 1931, by Messrs. T. F. Hickerson, F. H. Constant, W. N. Downey and E. C. Hartman; September, 1931, by Messrs. Thomas C. Shedd, David M. Wilson, and Marshall G. Findley; and November, 1931, by Messrs. George E. Large, and Sophus Thompson and R. W. Cutler.

[31] Engr., Bridge Dept., C. P. Ry., Montreal, Que., Canada.

[31a] Received by the Secretary November 27, 1931.

[32] *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 167.

in which, p is the area of the $\frac{M}{EI}$ = diagram for $M = 1$ at the left support of the beam with unit span, having E and I (minimum) $= 1$; q is the corresponding area for $M = 1$ at the right support, with the same understandings; u is the distance from the left support to the left characteristic point when $l = 1$; and v is the distance from the right support to the right characteristic point when $l = 1$, and α and γ represent the numerators of the moment-distribution factors for the beams, AB and BA, the sum of such θ-coefficients for the members meeting at one point constituting the denominator.

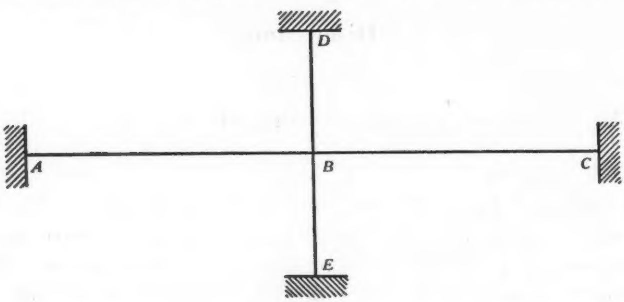

FIG. 54.

Thus, in Fig. 54, let the equations for the members be,

$$M_{BA} = \gamma\,\theta_B + \delta\,\theta_A \qquad (94a)$$

$$M_{BC} = \eta\,\theta_B + \kappa\,\theta_C \qquad (94b)$$

$$M_{BD} = \nu\,\theta_B + \epsilon\,\theta_D \qquad (94c)$$

$$M_{BE} = \tau\,\theta_B + \psi\,\theta_E \qquad (94d)$$

Then, locking the ends remote from their point of juncture, θ_A, θ_C, θ_D, θ_E are all equal to zero and,

$$M_{BA} = \gamma\,\theta_B \qquad (95a)$$

$$M_{BC} = \eta\,\theta_B \qquad (95b)$$

$$M_{BD} = \nu\,\theta_B \qquad (95c)$$

$$M_{BE} = \tau\,\theta_B \qquad (95d)$$

and the moment-distribution factors are:

Beam	Moment distribution factor	Beam	Moment distribution factor
$BA =$	$\dfrac{\gamma}{(\gamma + \eta + \nu + \tau)}$;	$BC =$	$\dfrac{\eta}{(\gamma + \eta + \nu + \tau)}$
$BD =$	$\dfrac{\nu}{(\gamma + \eta + \nu + \tau)}$;	$BE =$	$\dfrac{\tau}{(\gamma + \eta + \nu + \tau)}$

Furthermore, when such general equations are combined with the particular equation for a column, PQ, fixed at the base, Q, $M_{pa} = 4 \frac{EI}{h} \theta_p$, care must be taken that the moment of inertia used, is expressed in terms of that used as unity for the entire frame; otherwise, the term, $\frac{4\,EI}{h}$, cannot be compared properly with θ-coefficient of the beam equations, since that coefficient includes the moment of inertia of the beam. For the same reason, if the inertia moments at the centers of various haunched beams are different, the θ-coefficients must be modified accordingly.

Finally, the carry-over factors will be: From A to B, $\frac{p\,u}{q\,(1-v)}$; and, from B to A, $\frac{q\,v}{p\,(1-u)}$.

The writer prefers the convention in signs supported by the majority of the discussers—all clockwise moments positive.

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DISCUSSIONS

CONSTRUCTION OF LA OLA PIPE LINE, IN CHILE

Discussion

BY W. B. SAUNDERS, M. AM. SOC. C. E.

W. B. SAUNDERS,[4] M. AM. SOC. C. E. (by letter).[4a]—Since the completion of La Ola Pipe Line in November, 1926, operating history over a period of five years has answered in part the discussions by Messrs. Nelson, Thackwell, and Rudolph.

The general interest which would be accorded data concerning the carrying capacity of La Ola Pipe Line is mentioned by Mr. Nelson. For purposes of efficient operation and observation daily weir readings have been recorded at two different points on the line since December, 1926, from which some interesting data have been accumulated and, without going into minute details, the following observations have been made.

The section of pipe from the intake at La Ola Dam through to La Cabra Tunnel is that best adapted for experimentation and observation. In this section the hydraulic grade is 1.5 per 1 000 when the reservoir is full, and the pipe is of a uniform diameter throughout, with an average of 36.83 in. between male and female plates.

At the beginning of operation, when the pipe was new and the asphalt coating was very smooth, the records show a capacity of 28.5 cu. ft. per sec., but this capacity fell off rapidly at the beginning of operation to about 27.0 cu. ft. per sec. By July, 1929, the discharge had decreased to 25.5 cu. ft. per sec., and an examination of the inside of the pipe showed a nodular incrustation forming, which materially increased the friction and reduced the inside diameter of the pipe.

After some experimenting a "go-devil" was devised which successfully cleans the pipe. The main frame of this apparatus is of wood, bound at the joints and contact points with steel plates, to which are attached steel

NOTE.—The paper by W. B. Saunders, M. Am. Soc. C. E., was published in September, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: February, 1931, by Messrs. Ernest B. Nelson, H. L. Thackwell, and William E. Rudolph.

[4] Hydr. Engr., Andes Copper Min. Co., Barquito, Chañaral, Chile.

[4a] Received by the Secretary November 27, 1931.

brushes. The total weight and displacement is such that it will just float in water. It is moved through the pipe by the pressure head of water exerted on the circular disks. These disks are of such size, and their lengths are such, that the "go-devil" will easily pass through the flat curves in the pipe line. In theory, the disks form sufficient obstruction to allow the building up of about 10 lb. per sq. in. pressure head over their areas before the openings around them will pass the full flow of the pipe line. In practice, it is not fully known what pressure is exerted to move this apparatus, but when in use, the flow capacity of the line is reduced about 22%, and the "go-devil" moves at an average rate of about 2.5 ft. per sec., which approximates the velocity of the water at the reduced flow capacity.

In operation, this apparatus takes a rotary motion and successfully brushes out the incrustations from the line. By using it every six months the line capacity is maintained at 27.0 cu. ft. per sec., or more.

Toe coefficient of roughness, C (Hazen and Williams' formula), as applied to this pipe line and its operation, has been observed to be as indicated in Table 6.

TABLE 6.—QUANTITIES, VELOCITIES, COEFFICIENTS, AND CONDITIONS OBSERVED AND COMPUTED FROM LA OLA PIPE LINE OPERATION

Observed quantity of flow, in cubic feet per second	Computed velocity, in feet per second	Coefficient (Hazen and Williams), C	Condition
28.5	3.83	115	New pipe
27.5	3.73	112	Immediately after cleaning
27.0	3.66	110	60 days old and after cleaning
26.5	3.59	108	2 years old
25.5	3.46	104	2½ years old
24.5	3.32	100	Not yet reached

Mr. Thackwell presents a hydrograph (Fig. 13) which indicates the possibility of obtaining a continuous flow of 29 cu. ft. per sec. of water, provided 4 000 acre-ft. of storage might be made available at the head of the pipe line. The topography of the country and the available dam sites do not permit the use of such a hydrograph, within a reasonable cost. The present (1931) storage or regulation at La Ola Dam is about 25 acre-ft., and the site of the dam does not permit this quantity of storage to be increased at that point. The character of the country is such that an appreciable quantity of storage at any other available point would be prohibitive in cost. However, after two years of operation, additional water was added to the La Ola supply by the construction of 4.2 miles of pipe line, ranging in diameter from 14 to 18 in., which diverts 12 cu. ft. per sec. of water from the Juncal River (see Fig. 2) and puts this water into a small stream leading into the head of La Ola Vega. After losses from seepage and evaporation, which are very large, this quantity of water is sufficient to regulate the flow of La Ola River at the intake of the pipe line so as to release a larger quantity than that which will pass through the pipe.

Mr. Rudolph's discussion and tabulated costs are interesting and show efficient management of that work. It is apparent that living and labor conditions must have been very similar on the two jobs and that the cost of labor was much the same.

A direct comparison of other costs between the two jobs is difficult, except between the cost of steel pipe purchased in the United States. The small difference in cost for this pipe is easily accounted for by a probable difference in ocean and rail freight charges and a different date of purchase. The comparison of these costs with those of riveted steel pipe fabricated on the La Ola Line are all in favor of the local fabrication. On the large tonnage of pipe used in this line the saving effected by local fabrication was approximately $500 000.

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DISCUSSIONS

TESTS OF BROAD-CRESTED WEIRS

Discussion

By James G. Woodburn, Assoc. M. Am. Soc. C. E.

James G. Woodburn,[11] Assoc. M. Am. Soc. C. E. (by letter).[11a]—The able discussions which have been called forth by this paper are greatly appreciated. These discussions emphasize the varied and complex nature of the phenomena connected with flow over a broad-crested weir. As Professor Woodward points out, this type of weir presents many fascinating problems for study. Among the infinite variety of crest forms there must be certain designs which best combine a simple and constant head-discharge relation with minimum obstruction to flow and with ruggedness of construction and durability of crest. The ability of a broad-crested weir to operate at least 50% submerged (possibly 80% in some cases, as mentioned by Mr. Parshall) with practically no change in coefficient, gives it an advantage over a sharp-crested weir where head is at a premium; and the fact that it can be built without sharp edges to wear round and thus materially alter the discharge coefficient, recommends it for use in measuring the discharge of débris-carrying streams.

As a closing discussion of this first part of an extended study, the writer offers the following summary of the characteristics of flow over a broad-crested weir and of the outstanding points which require further study and explanation.

Since any weir not sharp-crested is, in effect, a short open channel, most of the phenomena of flow in open channels can also be observed in flow over broad-crested weirs. The chief cause of complications in broad-crested weir studies, as in any open-channel studies, is that the simple conditions of uniform, parallel flow are almost always absent. The problem is usually one of accelerated or retarded flow, of back-water or down-drop curves, and of wave formation, a complete explanation of which leads far into physics and hydrodynamics.

Note.—The paper by James G. Woodburn, Assoc. M. Am. Soc. C. E., was published in September, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: October, 1930, by H. W. King, M. Am. Soc. C. E.; December, 1930, by Messrs. Sherman M. Woodward and David L. Yarnell; February, 1931, by Messrs. Boris A. Bakhmeteff and D. D. Curtis; and September, 1931, by Messrs. Erik G. W. Lindquist and Ralph L. Parshall.

[11] Associate Prof. of Hydr. Eng., State Coll. of Washington, Pullman, Wash.

[11a] Received by the Secretary December 7, 1931.

In considering the broad-crested weir as a short open channel, it is apparent that friction losses in flow across the weir must not be neglected. These losses reduce the discharge below the theoretical, frictionless maximum by an amount which depends on the location of critical depth.

Critical depth, or depth at maximum discharge for a given height of energy gradient, always occurs at some point in non-submerged flow over a broad-crested weir. It may be expressed both as a fraction of the total head and also as a function of the discharge. From these relations the discharge at the point of critical depth, which is the largest possible discharge for the channel in question, can be expressed as a function of the total head. It should be noted that the head referred to in the case is the total head at the section of critical depth.

The location of critical depth in flow over a broad-crested weir is usually not easy to determine. Two cases arise for consideration.

1.—Parallel Stream Lines.—In this case the relations between depth, discharge per unit width of channel, and total head, are: $D_c = \frac{2}{3} H_a$, $D_c = \sqrt[3]{\frac{Q_1^2}{g}}$, and $Q_1 = 3.087\, H_a^{\frac{3}{2}}$ (English units).

The total head is represented by the elevation of the energy gradient above the stream bed, and with parallel flow it is obtained by adding the velocity head, $\alpha \frac{V^2}{2g}$, to the elevation of the water surface, since the pressure head at any point in the cross-section of the stream equals the height of the column of water above the point.

2.—Vertically Curving Stream Lines.—Since there is seldom any point at which flow over a broad-crested weir is parallel, it is necessary to consider the effect of vertical curvature of the stream lines, and to modify the equations for discharge and critical depth to take care of the change in pressure caused by centrifugal forces. The effect of vertical curvature of the stream lines has been treated fully in the discussions by Professor Bahkmeteff and Dr. Lindquist. Another treatment of this subject, applied particularly to down-drop curves, was published in 1929 by Dr. P. Boess, in reporting on some experimental studies in the Karlsruhe Laboratory.[12]

The curvature of the stream lines may be either downward or upward with respect to the stream bed. If they curve down toward the stream bed, centrifugal action reduces the pressure at all points in the cross-section below true hydrostatic pressure. Dr. Boess found by tests that it could usually be assumed with sufficient accuracy that this reduction in pressure is in straight-line variation with the depth. Since friction losses in accelerated flow are usually small, the energy gradient is very nearly horizontal. Thus, a reduction in pressure head causes an increase in velocity head, with resulting increase in velocity, in discharge, and in critical depth, for a given height of energy gradient.

[12] "Berechnung der Abflussmengen und der Wasserspiegellage bei Abstürzen und Schwellen," *Wasserkraft und Wasserwirtschaft*, 1929, No. 2–3. The writer's translation of this work has been filed for reference in Engineering Societies Library, 33 West 39th Street, New York, N. Y.

With downward curving stream lines, therefore, D_c is larger than $\frac{2}{3} H_a$, and Q_1 (max.) is larger than $3.087 H_a^{\frac{3}{2}}$. Conversely, also, with upward curving stream lines, D_c is less than $\frac{2}{3} H_a$ and Q_1 (max.) is less than $3.087 H_a^{\frac{3}{2}}$.

The application of these relations to a study of the coefficients of discharge of broad-crested weirs may be made through the equation for maximum unit discharge,

$$Q_1 \text{ (max.)} = C H_a^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (55)$$

The largest possible value of C in this equation is equal to, greater than, or less than, 3.087, depending on whether the stream lines are parallel, downward curving, or upward curving with respect to the stream bed. Furthermore, H_a in Equation (55) is the total head at the section of critical depth. Because of the continual loss of energy in a flowing stream, if the head at any point up stream from the point of critical depth is used in the equation, $C = \frac{Q_1}{H_a^{\frac{3}{2}}}$, the value obtained for C is less than the maximum value by an amount depending on the loss of head between the two points. This loss of head varies with the distance, with approximately the square of the velocity of flow, and probably also with the number of waves, between the two points.

Herein lies an explanation of why the coefficient of discharge of some of the weirs tested increased as the head increased, while that of other weirs decreased. In the former class are all the weirs with level crest. Critical depth occurred well down the crest at or below a point of approximately parallel flow, signifying a limiting value of C of about 3.087. Movement of critical depth up stream and decrease in the number of waves with the increase in head tend to reduce the losses between the point of measurement on the weir and the location of critical depth. Increasing velocity, on the other hand, tends to increase the loss. The increase in the coefficient with increasing head indicates that the resultant of the three factors was a reduction in the loss of head.

The writer's weirs with crests sloping slightly down stream, and Professor Webb's weirs which sloped slightly both ways from a crest line, show the coefficient decreasing as the head increases. Critical depth occurred usually near the entrance or highest point, and was either nearly constant in location or moved down stream as the head increased. This increase in distance and the increasing loss of head due to increasing velocity combine to cause a decrease in the coefficient.

Of course, the limiting value of C is not necessarily constant for a given weir, but may change with the head. This change becomes more pronounced as the down-stream slope of the crest increases. Thus, the pressure on the round-crested weir described by Mr. Yarnell (Figs. 14 and 16) is evidently reduced further below true hydrostatic pressure as the head increases, with resulting increase in C.

This reasoning with regard to the weir coefficient suggests the possibility of designing a broad-crested weir having a constant coefficient. The general requirements apparently are that there be approximately parallel flow at some point on the weir so that the coefficient shall have a definite limiting value; and that the resultant loss between the point of measurement of head and the location of critical depth shall be constant. It appears to the writer that the design of a weir of constant coefficient presents fewer difficulties than the design for which critical depth occurs at the same location at all heads.

Further progress in the study of flow over broad-crested weirs requires first an accurate experimental determination of the energy gradient. The height of the energy gradient at any point can be computed accurately only when the distribution of pressures and velocities over the cross-section is known. The accumulation of data regarding pressures and velocities on many different forms of weir crest is to be desired.

With the energy gradient drawn, and the pressure distribution known, the position of true critical depth can be located and the variation of this position can be studied with varying head and with different shapes of crest.

Further study is needed of the coefficient of discharge of broad-crested weirs of many different designs. An interesting question is to determine the boundary point in design between weirs which show a coefficient increasing as the head increases and weirs which show a coefficient decreasing as the

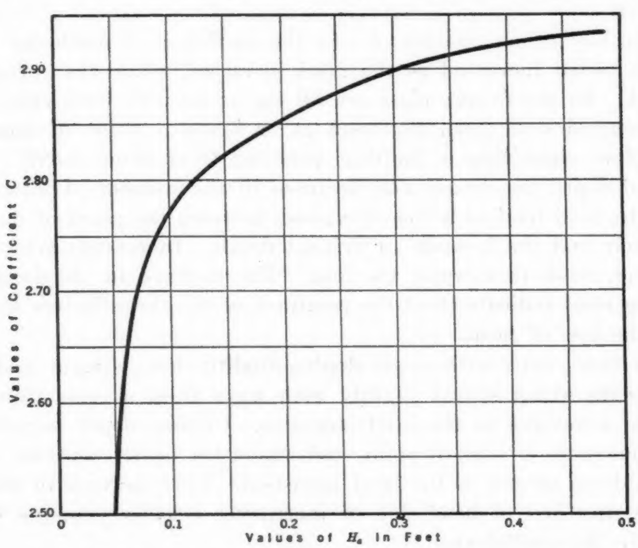
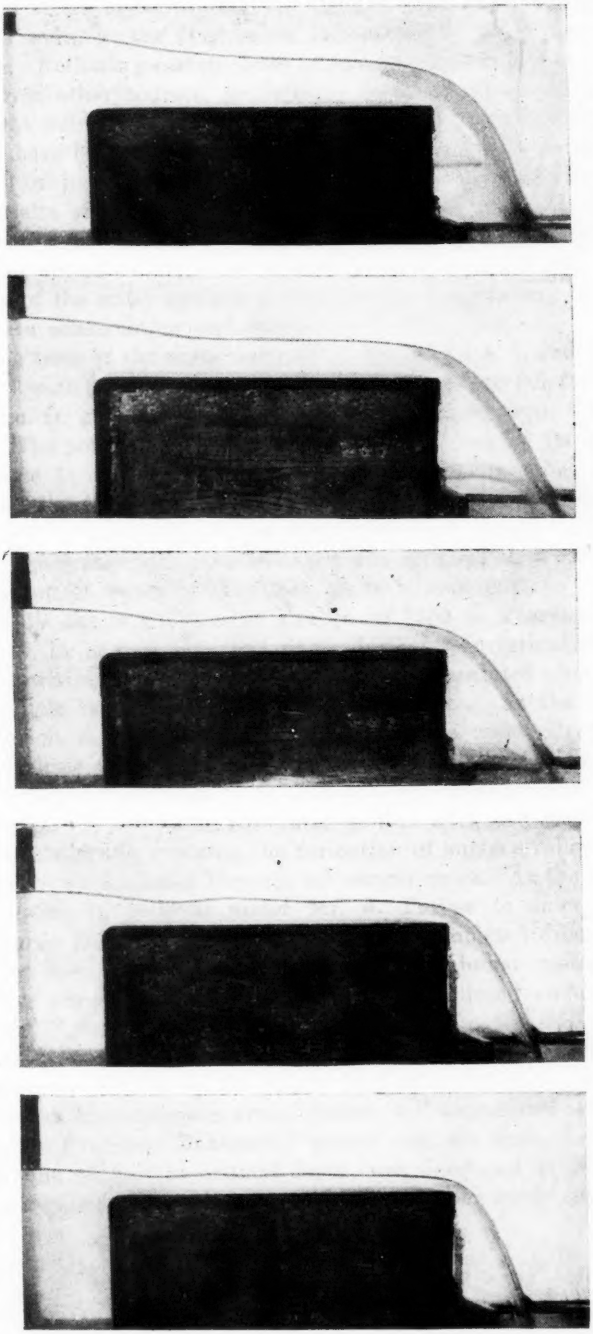


FIG. 39.—VARIATION OF WEIR COEFFICIENT, C, WITH TOTAL HEAD, H_a FROM EQUATION, $C = \frac{Q_1}{H_a^{\frac{3}{2}}}$

head increases. Another point to be studied is the effect on the coefficient of small changes in the slope of the weir crest. There are also few data available regarding the effect of various degrees of rounding of the entrance to the weir.



$Q_1 = 0.938$
$H_a = 0.468$
$C = 2.93$

$Q_1 = 0.604$
$H_a = 0.351$
$C = 2.91$

$Q_1 = 0.394$
$H_a = 0.265$
$C = 2.88$

$Q_1 = 0.269$
$H_a = 0.207$
$C = 2.86$

$Q_1 = 0.168$
$H_a = 0.153$
$C = 2.82$

FIG. 40.—FLOW OF WATER OVER BROAD-CRESTED WEIR.

The writer is continuing the experimental study of the flow over broad-crested weirs in the Hydraulics Laboratory of the State College of Washington. Following certain lines of investigation suggested in the discussion, and from other sources, preliminary tests are being made of a number of different weir crests in a glass-walled flume, 1 ft. wide. The first three weirs tested have been made of wood, with level crest 2 ft. broad, and 4 in., 8 in., and 12 in. in height, with rounded entrance. Comparison will be made with the results of the tests described in the writer's paper in order to study the application of the laws of similarity. A number of other crest designs will then be tested to study in a general way the coefficient of discharge and profile of the water surface so that certain designs may be selected for more accurate construction and study.

The tests of the three weirs (2 ft. broad and 4, 8, and 12 in. high, respectively) with level crest show that for heads up to 0.5 ft. and discharges up to 1 cu. ft. per sec., the weir coefficient increases with the head. (See Fig. 39.) The point of measurement of the head was 2.0 ft. up stream from the entrance to the weir. Practically the same curve has been obtained for each of the three heights of weir. Fig. 40 shows conditions of flow over the weir 12 in. high. The crest was level, 2.0 ft. broad and 1.0 ft. high, in a flume 0.999 ft. wide. The entrance was rounded on a radius of 0.1 ft. The formation of waves at the lower heads is apparent.

While the writer was in Europe in 1930 as Freeman Scholar from the Society, he was on the alert particularly for investigations of broad-crested weirs, critical depth, hydraulic jump, and associated phenomena. He found very little work being done along these lines. At the Technical Institute of Zurich, a candidate for a doctor's degree under Professor Robert Dubs was making some tests of a weir about 6 ft. broad with a sloping entrance in a channel 1 m. wide, with a view to the measurement of irrigation water in Egypt. At the same Institute, another advanced student under Professor Meyer-Peter was studying the formation of surface rollers and the hydraulic jump in the discharge through submerged gates. At the University of Delft, tests were in progress under Mr. J. Thijsse to show the nature of the discharge to be expected over the partly completed dikes of the Zuyder Zee during flood tides. These dikes are really broad-crested weirs of a cross-section somewhat similar to that of the railroad embankments mentioned by Mr. Yarnell.

Aside from these investigations, the writer found no tests on this subject in progress in any of the thirty hydraulics laboratories visited. However, European hydraulicians are of course well acquainted with the problem. In fact, as Professor Bahkmeteff points out, the theory of the two stages of flow, and of flow at critical depth, was developed at nearly the same time by a number of investigators, working independently, in different countries.

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DISCUSSIONS

A DISCHARGE DIAGRAM FOR UNIFORM FLOW IN OPEN CHANNELS

Discussion

By Murray Blanchard, M. Am. Soc. C. E.

Murray Blanchard,[13] M. Am. Soc. C. E. (by letter).[13a]—The writer is gratified and pleased with the discussions of his paper. They contain suggestions to simplify the procedure and indicate numerous uses that have been made of the principles and theories involved. The arguments relate to the different features or steps in the derivation of the discharge diagram and its uses, and will be treated separately.

The Base Curve.—Mr. Macphail proposes a substitute curve of K, referred to stage and discharge, to be used as a base from which, by Equation (2), are obtained the curves of the discharge diagram (Fig. 7). The base curve is derived from Equation (2) transposed to,

$$K = \frac{Q}{F^{0.5}} \qquad (15)$$

General Pillsbury proposes the same curve in different terms. His is a curve of 1-ft. fall (Fig. 9), obtained by making F unity in Equation (4), whence,

$$Q_x = \frac{Q}{F^{0.5}} \qquad (16)$$

and $K = Q_x$.

Mr. Perry uses the same relation (Equation (2)), as the others and obtains a diagram of K curves referred to discharge and fall. These are, in effect, a series of base curves, but the procedure is lengthened by the necessity of determining the values of (A), (C), and (R), in order to obtain a discharge corresponding to a given stage and slope.

Mr. Matheson does not state, but in correspondence with the writer he has revealed, that his method, developed about 1915, makes use of the 1-ft.

Note.—The paper by Murray Blanchard, M. Am. Soc. C. E., was published in January, 1931, *Proceedings*. Discussion on the paper has appeared in *Proceedings* as follows: May, 1931, by Messrs. Lynn Perry, J. B. McPhail, G. B. Pillsbury, Lynn Crandall, and Benjamin E. Jones; September, 1931, by A. J. Matheson, Esq.; and November, 1931, by Messrs. L. Standish Hall, and Ray E. Mackenzie.

[13] Cons. Engr., Chicago, Ill.

[13a] Received by the Secretary December 1, 1931.

fall curve (Fig. 9), and he calls it the "unit-fall" method. His suggestion to plot the resulting unit-fall discharges derived from each observation, in order to note any inconsistencies, is not considered by the writer as a necessary step because at any elevation all the observations, if they gave true discharges, would reduce to the same quantity. Comparison of these figures will enable one to detect any inconsistencies and will permit the elimination of abnormal observations. The arithmetic mean will give the best determination of a point on the base curve. The points plotted to determine the curve of Fig. 9, as well as the points for the base curve, Fig. 6, are groups or arithmetic means.

The K and the unit-fall base curves are the same as the writer's 1.0-ft. curve of fall in Fig. 7 and, as a slightly shorter operation, he would suggest the substitution of Fig. 9 for Fig. 7. The resulting discharge diagram will not be effected.

The Chezy Formula.—The references of Mr. Perry to the Kutter formula do not appear to be pertinent to the method adopted in the derivation of the discharge diagram for the reasons stated in the paper, that, "for a single stage under consideration the factors, (A), (C), (R), and (L), are constant" and are eliminated in the derivation of Equation (4) or Equation (5) from Equations (2) and (3). If Mr. Perry chooses to use his diagram (Fig. 8), the writer would suggest, for the determination of C, the Manning formula or the Kutter formula with the s terms omitted, which gives practically the same results, as proposed[14] by H. W. King, M. Am. Soc. C. E.

In the Chezy formula, $V = C\sqrt{RS}$, the factors, V, C, R, and S, are the means for the reach under consideration. Because the discharge is often measured at an end station, the velocity or hydraulic radius at this end station is frequently used as if it were the mean for the reach. Such an approximation will ordinarily give satisfactory results, as in Mr. Jones' method of correcting discharge for changing stage.

Normal Discharge.—This is a term of general usage that serves a useful purpose in the investigation of certain stream flows; but it is not for the general application that is sometimes given to it. It has been explained[15] as the discharge at a stage which would be caused by a slope that is the average of all the slopes that obtained for the period of record. Such a slope may never occur at the stage under consideration, especially if this is a flood stage. Such discharges should be labeled "normal," and the period of record and average slope used should be noted. Mr. Jones states, in reference to the Hall-Pierce method, which is based on Equations (9) and (10) how "the true discharge was obtained by multiplying the 'normal' discharge by the $\sqrt{z}$ factor taken from the table." This implies that there is only one true discharge for a stage, whereas, there may be a very large range of true discharges depending on the slopes that obtain. In other words, the Hall-Pierce method is applicable only on streams to which the rating curve applies and not where there is such a range of slopes at various stages that

[14] "Hydraulics," by King and Wisler, 1922 Edition, p. 190, and "Handbook of Hydraulics," by Horace W. King, Second Edition, 1929, p. 261.

[15] *Water Supply Paper No. 345,* U. S. Geological Survey.

the discharge diagram is necessary. The situation on the Tennessee River, at Chattanooga, which the "normal" stage method was designed to meet, is one in which the discharge diagram can be used to obviate the necessity of approximations.

Reference to Other Than Mean Stage.—The method of Mr. King, to which Mr. Jones refers, and that of Mr. Crandall, wherein discharges are referred to the stage at one station instead of the mean elevation of the reach, are approximate methods, but with a considerable field of adaptability. The writer has similarly used discharge diagrams, referred to a station at the end of a reach, to advantage in studying flood flows of the Lower Illinois River. The approximation comes in the assumption that the stage at other than the mean station varies as the mean stage. This assumption does not hold in a case in which there is artificial regulation above and below a reach, and hence it does not always follow that, for a single (other than mean station) stage under consideration, the factors (means for the reach), (A), (C), (R), and (L), are constant.

Application to Back-Water Computations.—Mr. Mackenzie has made application of the stage-slope-discharge relation to the determination of the back-water curve. He applies it to the condition of a stream in which there is only one discharge corresponding to a stage. A calculation can be made that would be applicable to all open channels, by a "cut and try" method with the discharge diagram. Thus, referring to the Chicago Sanitary District Canal, for which there is no established rating curve—and upon which the experiments were made that were used in this paper to demonstrate the derivation of the discharge diagram—the following is an example, using Mr. Mackenzie's notation and Figs. 11 and 7.

The problem is as follows: While flowing 8 000 cu. ft. per sec. in the reach from Willow Springs (B) to Lemont (A), the water surface is raised to — 4.85 at (A). What will be the resulting water surface elevation at (B)? The solution is:

	First trial	Second trial
Assume $F_x =$	1.1	1.0
From Fig. 7, or Table 3, $h_m =$	— 5.04	— 4.35
$\frac{F_x}{2} =$	0.55	0.50
$h_m - \frac{F_x}{2} = h_b =$	— 5.59 (too low)	— 4.85 (O. K.)

From the second trial, which checks for 8 000 cu. ft. per sec., the assumed fall, added to h_a, gives — 3.85, the required elevation.

Rating Table.—As stated by Mr. Mackenzie a rating table may be desirable. In Table 3, note that, in each stage column, the discharges, 8 000 cu. ft. per sec., are the points where the base curve (Fig. 6) intersects the "fall" lines at 0.5 to 1.2 ft. The stages thus obtained, — 0.90 to — 5.73, were used as convenient elevations along which to transpose the discharge-stage-slope relation. If Fig. 9 is used instead of Fig. 6 for a base curve it would be more convenient to use the even foot stages, — 1.0 to — 6.0.

Discharge Diagram.—From the variety of practical uses that can be made of the discharge diagram it would appear, generally, to be worth the effort to develop. General Pillsbury has suggested that the base curve (Fig. 9), is sufficient and that, instead of developing the discharge diagram, he would calculate from the curve, for given data, the discharges as required. The other extreme is taken by Mr. Mackenzie who would construct the discharge diagram and from it make a rating table. The writer concurs with him that "the form taken should be that which best suits the immediate purpose." From the Chicago Sanitary District Canal experiments the writer

TABLE 3.—RATING TABLE FOR DISCHARGE DIAGRAM (FIG. 7)

Fall, in feet	Discharge at Stage (Elevation Below Chicago City Datum), in Cubic Feet per Second							
	—0.90	—1.59	—2.28	—2.97	—3.66	—4.35	—5.04	—5.73
0.1......	3 580	3 260	3 020	2 830	2 670	2 530	2 410	2 300
0.2......	5 060	4 620	4 280	4 000	3 780	3 580	3 420	3 260
0.3......	6 200	5 660	5 240	4 900	4 620	4 380	4 180	4 000
0.4......	7 260	6 540	6 050	5 660	5 340	5 060	4 830	4 620
0.5......	8 000	7 310	6 760	6 320	5 960	5 660	5 400	5 140
0.6......	8 770	8 000	7 400	6 940	6 540	6 200	5 900	5 660
0.7......	9 460	8 650	8 000	7 480	7 050	6 700	6 380	6 120
0.8......	10 110	9 240	8 540	8 000	7 550	7 160	6 820	6 540
0.9......	10 750	9 800	9 150	8 480	8 000	7 600	7 230	6 920
1.0......	11 300	10 300	9 640	8 950	8 440	8 000	7 620	7 310
1.1......	11 890	10 900	10 000	9 380	8 850	8 400	8 000	7 650
1.2......	......	11 300	10 480	9 800	9 250	8 750	8 350	8 000
1.3......	......	11 700	10 900	10 200	9 600	9 110	8 700	8 340
1.4......	......	......	11 300	10 600	9 970	9 470	9 020	8 650
1.5......	......	......	......	10 960	10 320	9 810	9 350	8 950
1.6......	......	......	......	11 300	10 660	10 120	9 650	9 250
1.7......	......	......	......	......	11 060	10 400	9 950	9 540
1.8......	......	......	......	......	11 300	10 700	10 220	9 800
1.9......	......	......	......	......	......	11 000	10 500	10 080
2.0......	......	......	......	......	......	11 300	10 800	10 320
2.1......	......	......	......	......	......	......	11 080	10 600
2.2......	......	......	......	......	......	......	11 300	10 820
2.3......	......	......	......	......	......	......	11 580	11 200
2.4......	......	......	......	......	......	......	......	11 300

produced,[16] several years ago, a diagram of discharge referred to the elevations of the end stations. Mr. Matheson has developed a stage-slope-discharge relation of this form by the use of unit-fall base curves and has made extensive application. His suggestions as to precautions to be taken in making discharge measurements for this purpose are pertinent.

[16] "Hydraulics of Chicago Sanitary District's Main Channel," *Journal*, Western Soc. of Engrs., September, 1920, opp. p. 506.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

RUN-OFF—RATIONAL RUN-OFF FORMULAS

Discussion

BY CLARENCE S. JARVIS, M. AM. SOC. C. E.

CLARENCE S. JARVIS,[35] M. AM. SOC. C. E. (by letter).[35a]—The authors have made a clear exposition of many fundamental principles and considerations related to stream flow, and have attempted a somewhat rigorous analysis, evaluation, and correlation of various factors for practical use. So much of the textual treatment is commendable and beyond challenge that it is with reluctance that the writer registers minor exceptions or divergent views.

Having deliberately and repeatedly approached the subject of run-off, together with the derivation and critical tests of the best-known formulas, with an open mind, ready to accept or to reject any and all according to practical results within the fields for which data are available, he is unable to concur in either Considerations (1), (2), and (3), under "Empirical Formulas," or Statements (*d*) and (*e*) under "Inadequacy of Empirical Formulas." Do they not actually express the summation of all the influences mentioned in the paper under discussion, based on observed behavior instead of on *a priori* reasoning? While according an important place for the profitable application of the so-called rational method of analysis, one must likewise recognize its inherent limitations, disadvantages, and defects. Where detailed information is lacking on extensive water-sheds, the empirical formulas and high-water marks of past years will doubtless continue as the main reliance for purposes of design. Before the requisite mass of data can possibly be collected regarding physiography, topography, soils, channel characteristics, and rainfall habits, there will be many occasions for measuring the stream discharge and some part of the precipitation that preceded the gauging.

"The storm causing the greatest rate of discharge in a storm sewer will usually be the maximum rain lasting a length of time equal to the time of

NOTE.—The paper by R. L. Gregory and C. E. Arnold, Associate Members, Am. Soc. C. E., was published in April, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1931, by Messrs. Le Roy K. Sherman, Francis Bates, and John W. Raymond, Jr.; November, 1931, by Messrs. Reginald A. Ryves, G. S. Tapley, W. I. Hicks, John M. Kemmerer, Carl H. Reeves, Leonard L. Longacre, G. H. Hickox and Donald M. Baker; and December, 1931, by Albert R. Arledge, Assoc. M. Am. Soc. C. E.

[35] Prin. Hydr. Engr., U. S. Engr. Office, War Dept., New York, N. Y.

[35a] Received by the Secretary December 18, 1931.

concentration" is the original form of the statement published[36] by Anson Marston Past-President Am. Soc. C. E., in 1909, which has had great influence in guiding recent investigations. It is obvious that detailed analyses and tabulations are facilitated by the convenient assumptions that have been added as to "maximum uniform intensity" and, also, equal simultaneous distribution over large water-sheds during the time of concentration; but those assumptions are largely contrary to usual rainfall habits, especially over extensive drainage areas. Neither is the maximum rate of run-off necessarily the result of simultaneous contributions from all parts of the water-shed. It is more likely the result of an intense down-pour and concentration from a considerable portion, while much of the remaining area yields only a negligible run-off.

In keeping with the name usually accorded, the rational method should exclude such an expression for rainfall intensity as appears in Equation (2), which is capable of bringing forth absurdly high intensities for very short periods of time. Evidently, a constant should be added in the denominator to make it a safe factor as in Equation (45). Adolph F. Meyer M. Am. Soc. C. E., has recommended,

$$i = \frac{A\,t}{t + B} \qquad (92)$$

in which, both A and B are constants.[37] Furthermore, the introduction of N as a constant, based on a chance relation of certain variables within restricted range, may account for larger errors or irregularities than would be contemplated from Equation (18); for, in Equation (32), both the constant and the percentages are squared, and this would more than double the positive errors; for example, $1.10 \times 1.10 = 1.21$. Also, the varying channel sections, gradients, and velocities to be found in practically all natural drainage systems are quite contrary to the assumptions of the authors, who seem to be carrying the storm drainage conduits throughout the water-sheds under consideration. Otherwise, it is almost inconceivable that such velocities as 28.3 ft. per sec., or 19 miles per hour, would occur with the slope and discharge assumed in Example 3.

It seems to be contrary to experience that the instantaneous discharge at a given station should vary directly with the length of travel. Perhaps the total quantity bears such a relationship; but the increased time of travel affords opportunity to smooth out the flood crests, thus increasing channel storage, underflow, and period of high stage. If a linear dimension is desired instead of some function of the drainage area, the average width seems to have a more logical basis for adoption, as has been demonstrated[38] by Charles R. Pettis, M. Am. Soc. C. E.

The discharges of 1 006 and 1 069 sec-ft. from 2.02 sq. miles, as used in Examples 6 and 8, respectively, are equal to 7.1% and 7.5% on the Myers scale[39]; which the 59 740 and 62 003 sec-ft. from 500 sq. miles, as disclosed in

[36] Cyclopedia of Civil Engineering, Vol. VII, 1909, p. 305, "Sewers and Drains."
[37] "Elements of Hydrology, Precipitation," 1928, p. 103.
[38] "A New Theory of River Flood Flow," by Charles R. Pettis, M. Am. Soc. C. E.
[39] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 994.

Examples 7 and 9, represent 26.7% and 27.8%, respectively. Taking into account the time required for a reliable solution by small increments from individual areas, the many estimates, assumptions, and uncertainties involved, the acurate knowledge of the water-shed topography, geology, soil characteristics, and rainfall habits, that must be in hand for reliable results, and the involved factors and nomenclature required for the method presented in the paper, it appears that its special field for usefulness is in urban and metropolitan developments. To the writer the best of the empirical formulas seem better adapted to large areas and to the determination of run-off therefrom; and they seem to have fully as valid a claim to the descriptive adjective as has been shown for the "rational formulas" under discussion.

It is significant that the expression adopted by the Committee on Floods of the Boston Society of Civil Engineers,[40] readily reduces to a form comparable with the modified Myers formula,

$$Q = 10\,000 \times 80\% \sqrt{A} \quad \text{(93)}$$

thus ranging up to a rating of 80% for maximum floods during long periods of time.

Notwithstanding the implication so often expressed to the effect that such formulas do not take account of the shape and other variable characteristics of drainage areas, the estimated effects of such factors are readily incorporated in the percentage adopted. Thus, with ratios of length to width represented by 5, 2, and 1, the flood rating for a given water-shed and frequency of recurrence might be 50%, 60%, and 80%, respectively.

The platting of observed rainfall and run-off with time and average depths on the water-shed as co-ordinates, together with detailed studies of soil, slopes, vegetative cover, and geology along with physiography, should afford a fair index to probable stream flow from a given area, or from other water-sheds with similar characteristics, and there is a definite and important place for both the rational and the empirical methods of analysis.

[40] *Journal*, Boston Soc. of Civ. Engrs. (1920), Vol. VII, p. 47.

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DISCUSSIONS

DESIGN OF LARGE PIPE LINES

Discussion

By Messrs. Paul Bauman, and L. J. Larson

Paul Bauman,[30] M. Am. Soc. C. E. (by letter).[30a]—The ring support of large pipe lines, as outlined by the author, is a decided advance from a standpoint of statical clearness, as compared to the conventional saddle support. The reactions are concentrated and their points of application exactly defined. As the actions too are well established, the actual stress due to moments and shear in the continuous system necessarily must tally closely with the theoretical ones.

The maximum stresses in the pipe shell are likely to be higher with the ring supports than with the saddle supports of equal spacing, as in the latter case the distortion at the supports is less accentuated. The stress distribution in the latter case is too complex for an exact solution, however, and the types compare statically somewhat as a beam-and-girder slab and a flat-slab system of buildings.

Longitudinal shear is a maximum at the support where it combines with the longitudinal stress and the ring stress of the shell to a diagonal or principal stress. Referring to the author's numerical example, the shear, in pounds, on one side of the supporting ring is, $\frac{Q}{2} = \frac{317\,000}{2} = 158\,500$.

The maximum unit shear stress, s, due to $\frac{Q}{2}$, occurs in the plane of the neutral axis,

$$s = \frac{\frac{Q}{2} M_{sT}}{2\,t\,I} \qquad \text{(127)}$$

in which, M_{sT} is the statical moment of the upper half of the shell about the

Note.—The paper by Herman Schorer, Assoc. M. Am. Soc. C. E., was published in September, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1931, by Messrs. L. J. Mensch and W. P. Roop; December, 1931, by Messrs. Johannes Skytte, Donald E. Larson, Raymond J. Roark, and F. W. Hanna.

[30] Chf. Designer, Quinton, Code & Hill–Leeds & Barnard, Engrs., Consolidated, Los Angeles, Calif.

[30a] Received by the Secretary November 17, 1931.

horizontal axis of symmetry, and I, the moment of inertia of the shell about the horizontal axis of symmetry; but $M_{sT} = \pi\, r^2\, t \dfrac{\sin u}{u} = 2\, r^2\, t$; and $I = \pi\, t\, r^3$. Substituting in Equation (21), the maximum unit shear stress, in pounds per square inch, is, $s = \dfrac{Q\, r^3\, t}{2t^2\, \pi\, r^3} = \dfrac{Q}{2\, r\, \pi\, t} = 3\,365$. This value is checked by the author's Equations (5) and (9).

The maximum rim stress for $x \neq 0$, and a rigid ring is given by Equation (29) for,

$$\frac{b\,(f_{bx})}{d\,x} = 0 = f_{bo}\,\sqrt{2}\left[-\frac{e^{-\frac{x}{z}}}{z}\cos\left(\frac{x}{z}+\frac{\pi}{4}\right)-\frac{e^{-\frac{x}{z}}}{z}\sin\left(\frac{x}{z}+\frac{\pi}{4}\right)\right]$$

or,

$$\sin\left(\frac{x}{z}+\frac{\pi}{2}\right) = 0 \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(128)$$

which is the case for $\dfrac{x}{z} = \dfrac{\pi}{2}$; $\dfrac{3\,\pi}{2}$; .. etc.; and, since, $z = 0.78\,\sqrt{rt} = 3.025$, x for $\left. f_{bx} \right]_{\text{min.}}^{\text{max.}} = 4\tfrac{3}{4}$ in.; $14\tfrac{1}{4}$ in.; etc. For $x = 4\tfrac{3}{4}$ in.,

$$f_{bx}\ \text{min.} = 1.82\,\frac{pr}{t}\,\sqrt{2}\left[\frac{1}{4.81}\times\frac{-\sqrt{2}}{2} = -\,0.379\,\frac{pr}{t}\right]\ldots(129)$$

and for $x = 14\tfrac{1}{4}$ in.,

$$f_{bx}\ \text{max.} = \frac{4.81}{112}\times 0.379\,\frac{pr}{t} = +\,0.0163\,\frac{pr}{t}\quad\ldots\ldots\ldots\ldots(130)$$

Introducing numerical values, namely, $p = 100\,\dfrac{62.50}{144} = 43.30$ lb. per sq. in.; $r = 60$ in.; and $t = \frac{1}{4}$ in.; f_{bx} min. $= -\,3\,950$ lb. per sq. in.; and f_{bx} max. $= +\,170$ lb. per sq. in.

Neglecting the weight of the pipe, the ring stress at $x = 4\tfrac{3}{4}$ in. is not due to the full pressure, p, because that is partly carried longitudinally by beam action. To determine this beam load:

$$M_x = Ke^{-\frac{x}{z}}\cos\left(\frac{x}{z}+\frac{\pi}{4}\right)\ldots\ldots\ldots\ldots\ldots\ldots(131)$$

in which, $K = f_{bm}\,\dfrac{t^2}{6}\,\sqrt{2}$. Then,

$$\frac{dM_x}{dx} = Q_x = -\,\frac{Ke^{-\frac{x}{z}}}{z}\left[\sqrt{2}\,\sin\left(\frac{x}{z}+\frac{\pi}{2}\right)\right] = -\,\frac{Ke^{-\frac{x}{z}}}{z}\,\sqrt{2}\,\cos\frac{x}{z}\ .(132)$$

and,

$$\frac{d^2 M_x}{dx^2} = q_x = -\,\frac{K\sqrt{2}}{z}\left[-\frac{e^{-\frac{x}{z}}}{z}\sin\frac{x}{z} - \frac{e^{-\frac{x}{z}}}{z}\cos\frac{x}{z}\right]$$

$$= +\,\frac{2K}{z^2}\,e^{-\frac{x}{z}}\sin\left(\frac{x}{z}+\frac{\pi}{4}\right)\ldots\ldots\ldots\ldots\ldots\ldots(133)$$

for $x = 0$, $q_x = p$, which checks; and for $x = 4\frac{3}{4}$ in.,

$$q_x = \frac{2 \times 1.82 \frac{pr}{t} \times \frac{t^2}{6} \sqrt{2}}{0.607\, r\, t} \left[\frac{1}{4.81} \frac{+\sqrt{2}}{2} \right] \text{........(134)}$$

or, $q_x = 0.208\, p$. This gives a ring stress, $f_r = \dfrac{rp\,(1 - 0.208)}{t} = +\,8\,250$ lb. per sq. in. The unit shear stress, s, at $x = 4\frac{3}{4}$ in., based on $s = 0$ for $x = \dfrac{L}{2}$, is $s = \dfrac{12 \times 30 - 4.75}{360} \times 3\,365 = 3\,320$ lb. per sq. in.

In the plane of the neutral axis the longitudinal stress, $f_L = 0$. At $x = 4\frac{3}{4}$ in., therefore, $s = \pm\, 3\,320$ lb. per sq. in. $= \tau$; $f_{bx} = -\,3\,950$ lb. per sq. in. $= \sigma_x$; and $f_r = +\,8\,250$ lb. per sq. in. $= \sigma_y$.

The principal stresses are: $\sigma_{\max.} = \dfrac{-3\,950 + 8\,250}{2} + \frac{1}{2}\sqrt{4\,(3\,320)^2 + 12\,200^2}$ $= +\,2\,150 + 6\,940 = +\,9\,090$ lb. per sq. in.; $\sigma_{\min.} = -\,4\,790$ lb. per sq. in.; and $\tau_{\max.} = \pm\,6\,940$ lb. per sq. in.

If the unit shear stress, s ($= \pm\,3\,365$ lb. per sq. in.), is combined with the maximum rim bending stress, f_{bo} ($= \pm\,16\,580$ lb. per sq. in.), and if $\sigma_y = 0$: $\sigma_{\max.} = 8\,290 + \frac{1}{2}\sqrt{4\,(3\,365)^2 + 16\,580^2} = +\,17\,240$ lb. per sq. in., which is still only 74% of $f_L + f_{bo}$, which occurs on top of the shell at the supports. The latter is the critical tensile stress which is economically met by thickening the pipe locally as suggested by the author.

Equation (29), properly modified, permits determining the elastic line of the shell, for example,

$$\frac{EI}{1 - m^2} \frac{d^2 y}{d x^2} = M = K e^{-\frac{x}{z}} \cos\left(\frac{x}{z} + \frac{\pi}{4}\right) = K' \left[e^{-\frac{x}{z}} \cos\frac{x}{z} - e^{-\frac{x}{z}} \sin\frac{x}{z} \right] \text{.(135)}$$

in which, $K' = 1.82\, \dfrac{prt}{6}$. Furthermore,

$$\frac{EI}{1 - m^2} \frac{dy}{dx} = K' \left[\int e^{-\frac{x}{z}} \cos\frac{x}{z}\, dx - \int e^{-\frac{x}{z}} \sin\frac{x}{z}\, dx \right] + C_1 \text{...(136)}$$

or,

$$\frac{EI}{1 - m^2} \frac{dy}{dx} = K'\, z \left[e^{-\frac{x}{z}} \sin\frac{x}{z} \right] + C_1 \text{..............(137)}$$

for $x = 0$; $\dfrac{dy}{dx} = 0$; and $C_1 = 0$. Also,

$$\frac{EI}{1 - m^2}\, y = -\frac{K'\, z^2}{2} \left[e^{-\frac{x}{z}} \left(\sin\frac{x}{z} + \cos\frac{x}{z} \right) \right] + C_2$$

$$= -\frac{K'\, z^2}{2} \left[e^{-\frac{x}{z}} \sqrt{2} \sin\left(\frac{x}{2} + \frac{\pi}{4} \right) \right] + C_2 \text{................(138)}$$

for $x = 0$; $y = 0$; and $C_2 = + \frac{K' z^2}{2}$. Thus,

$$\frac{E\,I}{1 - m^2} y = \frac{K' z^2}{2} \left[1 - \sqrt{2}\, e^{-\frac{x}{z}} \sin\left(\frac{x}{z} + \frac{\pi}{4} \right) \right] \text{..........(139)}$$

in which, $\frac{K' z^2}{2} = \frac{1.11\, p\, r^2\, t^2}{12}$; and $I = \frac{t^3}{12}$.

Making the proper substitutions in Equation (139),

$$E\, y = \frac{p\, r}{t} \left[1 - \sqrt{2}\, e^{-\frac{x}{z}} \sin\left(\frac{x}{z} + \frac{\pi}{4} \right) \right] \text{...........(140)}$$

In using the numerical values as given in the author's example, a line as shown in Fig. 22, results. As x continues to grow, the term with $e^{-\frac{x}{z}}$ approaches zero and $E\, y$ approaches the ring distortion, r'_p. The loading, q_x, per unit area is shown in Fig. 22(b).

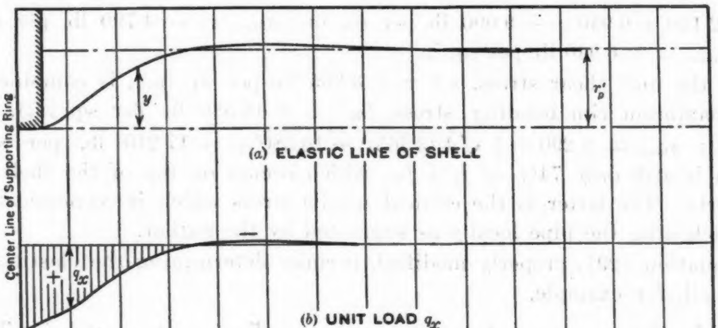

FIG. 22.

The line of pressure of the statically indeterminate supporting ring could readily be found graphically or analytically by means of the ellipse of elasticity, as previously shown by the writer.[31]

The problem is particularly simple, because the ellipse of elasticity in this case is a circle. Due to symmetry, it is sufficient to consider one-half only, either left or right. As a boundary condition, Reaction A or Reaction B must be tangent to the line of pressure.

A Check of Equations (31), (32), and (33).—The work of deformation is expressed by,

$$A = \int \frac{M^2}{2\, E I}\, dl \text{ ....(141)}$$

[31] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1672–1674.

and the condition of least work is,

$$\frac{\partial A}{\partial X_1} = \int \frac{M\left(\frac{\partial M}{\partial X_1}\right)}{EI} d\,l = 0 \quad \text{(142a)}$$

$$\frac{\partial A}{\partial Y_1} = \int \frac{M\left(\frac{\partial M}{\partial Y_1}\right)}{EI} d\,l = 0 \quad \text{(142b)}$$

and,

$$\frac{\partial A}{\partial M_z} = \int \frac{M\left(\frac{\partial M}{\partial M_z}\right)}{EI} d\,l = 0 \quad \text{(142c)}$$

Since $\frac{\partial M}{\partial X_1} = -y$; $\frac{\partial M}{\partial Y_1} = -x$; and $\frac{\partial M}{\partial M_z} = -1$:

$$\int (M_e - X_1 y - Y_1 x - M_z)\, y\, d\,l = 0 \quad \text{(143a)}$$

$$\int (M_e - X_1 y - Y_1 x - M_z)\, x\, d\,l = 0 \quad \text{(143b)}$$

and,

$$\int (M_e - X_1 y - Y_1 x - M_z)\, d\,l = 0 \quad \text{(143c)}$$

Since the X and Y axes go through the elastic center and are axes of symmetry, $\int y\, dl = 0$; $\int x\, dl = 0$; and $\int x\, y\, dl = 0$. Hence,

$$\int M_e\, y\, dl - X_1 \int y^2\, dl = 0 \quad \text{(144)}$$

or,

$$X_1 = \frac{\int M_e y\, dl}{\int y^2\, dl} \quad \text{(145a)}$$

$$Y_1 = \frac{\int M_e x\, dl}{\int x^2\, dl} = 0 \quad \text{(145b)}$$

and,

$$M_z = \frac{\int M_e\, dl}{\int dl} = 0 \quad \text{(145c)}$$

which check Equations (31), (32), and (33).

The total tension in the supporting ring is given by Equation (30), or, in general terms, by Equation (19). This is only approximately correct as the ratio between the outside and inside diameters of these rings may vary quite considerably from unity. The tension is then no longer uniformly distributed over the cross-section, but has a maximum intensity on the inside, and a minimum on the outside, of the ring.

If r_i is the inside radius; r_o, the outside radius; and $p' = pc + 2\,H$; then,

$$f_t = p'\,\frac{r_o^2 + r_i^2}{r_o^2 - r_i^2} = p'\,\frac{r_o^2 + r_i^2}{(r_o + r_i)A_r} \quad\ldots\ldots\ldots\ldots\ldots\ldots(146)$$

instead of, $f_s = \dfrac{p'\,r_1}{A_r} = 1\,020$ lb. per sq. in.

Let $pc + 2H = p' = \dfrac{A_r\,f_s}{r} = 204$ lb. per sq. in.; $r_i = 60$ in.; and $r_o = 72$ in. Then, the maximum tensile stress, using the same numerical values as before, is, $f_s = 204\,\dfrac{5\,200 + 3\,600}{1\,600} = 1\,125$ lb. per sq. in., or an increase of 10 per cent.

The author's formulas are also applicable to flanged pipes of small shell thickness.

For penstocks on steep inclines the weight component in the direction of the axis must be introduced. It produces a tensile stress if the pipe is anchored on top of the incline, which is a desirable arrangement. The expansion joint is then located either at the bottom of each run or immediately below the anchor point, depending on its type.

The elimination of uncertainties in the outer or inner stability of structures should be made an outstanding task by all designing engineers.

The author has again contributed to the clarification of statical systems and deserves the appreciation of his colleagues.

L. J. LARSON,[32] ASSOC. M. AM. SOC. C. E. (by letter).[32a]—This interesting paper gives a concise analysis of the action of a thin-walled large pipe supported by ring girders. It also indicates, to a striking degree, the advantage that may be gained by supporting a structure so as to carry the loads by direct, instead of bending, stresses. The dead-load stresses in a thin pipe improperly supported, may be greater than those due to the pressure for which the pipe is designed.

Referring to Equations (4), (5), and (6), it is interesting to note that all the unit stresses obtained by dividing each by the shell thickness, t, are independent of the thickness, since $\dfrac{w}{t}$ is the weight per unit volume of the material in the pipe.

Taking the weight of steel as 0.283 lb. per cu. in., the maximum values for the unit stresses are: $\dfrac{T_2}{t} = 0.283\,r$; $\dfrac{S}{t} = 0.283\,L$; and $\dfrac{T_1}{t} = \dfrac{0.283}{r} \times \dfrac{L^2}{4}$.

The unit ring stress obtained from Equation (4) is particularly striking when it is compared to the bending stresses in a similar pipe without ring girders, supported continuously along the bottom. Using the values assumed

[32] Research Engr., A. O. Smith Corporation, Milwaukee, Wis.

[32a] Received by the Secretary December 3, 1931.

by the author under "Example and Applications," the ring stress due to dead load is equal to: $\frac{T_2}{t} = 0.283 \times 60 = 16.98$ lb. per sq. in. If a pipe of the same dimensions were supported on a horizontal plane the maximum bending stress would be, approximately, $2.5 \frac{r^2}{t}$, or 36 000 lb. per sq. in. While it is true that this bending stress can be materially reduced by supporting the pipe on a larger area, or along two lines at the ends of the horizontal diameter, still the stress will be proportional to $\frac{r^2}{t}$, and large, thin pipes will be subject to high stresses and considerable deformation.

In the derivation of stresses in the pipe and the supporting ring, the author assumes that the shearing stresses in the pipe are applied to the ring. It would be of interest to know to what extent the results are affected if the ring is attached to the pipe only at intervals and, also, if the ring is not attached but fits the pipe snugly. The amount of slip or relative movement between the pipe and the ring, necessary to relieve all the shearing stresses may be very small, and if such movement occurs the ring will act as if it were unattached. The action of a loose ring may not be serious unless it changes the entire state of stress in the pipe and invalidates the equations developed. If the author has made any analysis based on a loose ring support, the results will be helpful.

In the application of the author's equations to tanks it would also be of interest to know to what extent the usual type of convex, "dished" heads act as ring girders. Such heads are not as stiff as flat plates in the radial direction, but they will withstand a considerable radial load without collapsing. As far as the head itself is concerned it could probably be used as a disk or ring girder in supporting a tank, but, it is a question whether the small movements or deformations permitted in the shell by such a support are sufficient to change the action of the entire shell in resisting the dead and live loads. If the author can show that the theory holds for tanks with various types of heads, his analysis will find many additional applications.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

SOIL MECHANICS RESEARCH

Discussion

BY MESSRS. JACOB FELD, AND JOHN R. JAHN

JACOB FELD,[21] ASSOC. M. AM. SOC. C. E. (by letter).[21a]—It is an ambitious program which Professor Gilboy outlines, and his reports of progress will be watched with great interest. It is a hopeful sign that he permits engineering problems to control the trend of research in soil mechanics, for only in that way can the practical observations collected in the field of construction be tied into the same network of facts.

The Wiegner method for mechanical analysis of finely pulverized materials brings out an important fact. The pressure of water containing suspended soil or any other material is greater than that of pure water. This fact applies even to such suspensions as freshly mixed concrete. The pressures are in direct proportion to the specific gravities of the suspensions. In other words, as far as engineering data are concerned, suspensions are liquids and follow the laws of liquids. Large-scale experiments by Professor E. B. Smith and by Shunk on concrete pressure, and work by Résal on clay fills, and by La Habra Dam Commission on mud, give results on various materials which check the theoretical values.

The tests developed for the various consistency limits may result in methods for distinguishing and classifying various soils. However, the writer cannot see any value in the definitions of the limits, as given in this paper. For example, under "Limits of Consistency," the author states: "All the limits are expressed in terms of water content, that is, weight of water per unit weight of dry soil." How is the unit weight of dry soils to be determined? What packing or loose condition can be taken as a standard, so that all experimenters will agree on a unit weight for any given sample?

A more serious objection, in addition to this one, applies to the definition of the shrinkage limit as given in Equation (1). It is a fundamental check in mathematical analysis of physical phenomenon that all equations must balance in dimensions. The definition requires the shrinkage limit value

NOTE.—The paper by Glennon Gilboy, Jun. Am. Soc. C. E., was presented at the meeting of the Structural Division, Boston, Mass., October 10, 1929, and published in October, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: December, 1931, by Messrs. J. C. Meem, and H. deB. Parsons.

[21] Cons. Engr., New York, N. Y.

[21a] Received by the Secretary November 25, 1931.

to be without dimension, that is, a pure number or ratio of weight to weight of equal volumes. The equation is weight minus volume, both divided by weight, and cannot truly represent the factor defined.

In studying the permeability of soils, it must be pointed out that Equation (3) is true only if: (1) The soil is entirely saturated before any reading is taken; (2) the flow through the soil sample is constant under constant pressure, that it is not clogging up or washing out; (3) the value of the constant, k, is independent of the pressure head, h; and (4) the temperatures of soil and water are the same. A very fine discussion of the variables controlling the flow of water through earth has been given[22] by John H. Griffith, M. Am. Soc. C. E.

The further data on the compression, tension, and shear values of soils add to the store of knowledge started by the very earliest experimenters in soil physics. It is these functions of a soil which distinguish it from the fluid and solid states of matter.

The author's statement under "Research in Soil Engineering," to the effect that, "corroborating previous evidence, the tests showed that the settlement under a given unit load, not too near the ultimate, is practically independent of the size of the loaded area," is interesting if it can be proved, even for sand. It is the writer's impression that, from recent observations of existing structures as well as from experiments, exactly the opposite has been the general trend. It is hoped that a detailed description of facts from which this conclusion is drawn, will be presented.

The work on lateral pressure on walls has been of special interest to the writer. The report of the experimental work performed for the New England Power Construction Company to which the author refers, is not complete enough to be conclusive. Apparently, from the results reported,[23] a great number of tests have been performed. It is hoped that a report of these tests will be published.

Professor Gilboy is to be complimented on his work in continuing Dr. Terzaghi's research, and for his clear exposition of the various lines of research in soils which he is covering.

JOHN R. JAHN,[24] ASSOC. M. AM. SOC. C. E. (by letter),[24a]—This interesting paper by Professor Gilboy shows the amount of careful study and the number of mechanical devices needed to produce complete information concerning the physical properties of soils. The writer was particularly interested in the short description of the hydrometer method of size analysis and feels that it was unfortunate that more was not written of the details of laboratory work and mathematical analysis. The author has wisely pointed out that accurate knowledge of size distribution is not vital to soil mechanics, since other factors are of such importance that size analyses give only a rough indication of the properties of the material. If he has found a method of making accurate analyses, however, it would be of interest to many laboratories that are

[22] "Physical Properties of Earths," *Bulletin 101*. Iowa Eng. Experiment Station.

[23] *Engineering*, May 30 and June 13, 1930.

[24] Civ. Engr., San Francisco, Calif.

[24a] Received by the Secretary December 5, 1931.

attempting to make determinations. It would also be of assistance to laboratories in other industries which depend on fine comminution, such as the sliming of ores, fine grinding of paints and pigments, cements, inks, etc.

The writer feels that the Bouyoucos method, with some of its improvements, is a useful tool for engineering information, but that it has inherent faults which will eliminate its use if accurate information is sought. These faults are fundamental, as will be brought out.

The hydrometer is shown at the left in the author's Fig. 2. It consists of an enlarged bulb portion with a weighted point and a thin stem with graduations, and at the level of submersion it registers the grammes of suspended matter per liter of liquid. A known weight of sample is thoroughly deflocculated and diluted to a known volume in the test cylinder. At various time intervals after the start of the analysis the hydrometer is lowered into the cylinder and the decreasing densities of the system give the data from which the size analysis of the sample is computed. Stokes' law[25] is used to determine the size of particle which will fall from the surface of the liquid to a critical depth point on the hydrometer in the observed time, t, and some quantitative method is used for the mathematical analysis.[26]

The following faults in the hydrometer method are apparent:

(*a*) With its irregular shape and conical point, the hydrometer does not have a practicable bottom level, to which to refer the amount of fall of particles.

(*b*) Due to its bulbous shape, too much emphasis is given to the density of the suspensoid opposite the region of the enlargement.

(*c*) The hydrometer must be withdrawn after each observation to avoid deposits on its upper shoulder. This causes turbulence, with an indeterminate translation of particles.

(*d*) The meniscus on both the cylinder and the hydrometer at the level of the liquid prevents close reading of the scale unless other means, such as external gauges, are used.

(*e*) Unless due correction is made, an error of considerable magnitude will remain in the analysis because of the settling out of particles smaller than the size determined by Stokes' formula, which have fallen beyond the hydrometer, from their initial subsurface positions, that is, from less than the full depth. If the semi-log graph of "the hydrometer reading" plotted against logarithm of "the elapsed time of settlement" is straight, the error will be constant throughout, but if the graph is curved, as is usual, the error will be variable in the different size classifications.

Corrections for all these faults, except (*c*), are being made in the laboratory of Charles H. Lee, M. Am. Soc. C. E., to adapt the hydrometer method to a more accurate size analysis of fine-grained engineering material, such as muds, soils, etc. Fault (*e*) is being corrected by a modification of Rubey's method[27] adapted to semi-log graphs; Fault (*d*) is avoided by arranging a

[25] *Transactions*, Cambridge Philosophical Soc., Vol. 8, p. 287, and Vol. 9, p. 8.

[26] "The Hydrometer Method for Making a Very Detailed Mechanical Analysis of Soils," by G. J. Bouyoucos, *Soil Science*, September, 1928.

[27] *Professional Paper 165A*, U. S. Geological Survey, p. 25.

mirror against which an index point is reflected; Fault (*c*) cannot be eliminated, but is minimized by very careful manipulation; and Faults (*a*) and (*b*) are corrected by a calibration method in which the errors are grouped and the correction factor ascertained by control tests, using a similar cylinder, but having a parting plane of separation at the lower limit of the hydrometer's subsidence. Its purpose is to establish the actual average density of the suspension at the instant of reading the hydrometer. The depth of the parting plane being known, it is possible to obtain a correction curve to convert hydrometer readings into average densities of a standard height of column at the same instants. These calibration corrections will have to be established for each class of material or until its character can be anticipated. The corrections are not constant over the range of the analysis, but have varied, according to texture, from 0 to 12%, within a period of a few minutes in the "run."

During the fall of 1924, the writer was seeking a method of making an accurate size analysis, using an adaptation of the Oden method[28] as developed at the University of Upsala, at Upsala, Sweden. This method weighs the accumulations of sediment on a scale pan, suspended from a balance and situated near the bottom of a test cylinder. At the Soil Technology Laboratory, of the University of California, Professor E. V. Winterer had assembled a well-designed apparatus for weighing the sediment and recording the time intervals for equal weight accumulations. It had close thermostatic control and was automatic in every particular after commencement of the test. The method was found to be grossly inaccurate in a very unusual way, in that the particles refused to fall vertically, but migrated away from the center line of the cylinder, that is, horizontal layers did not have uniform distribution of suspensoids. Errors of from 10 to 35%,[29] all of under-registry, were found, and experiments with cylinders of glass, metal, rubber, and wood and having electrical charges of from 1.5 to 30 000 volts impressed thereon in various combinations, were found to be ineffectual. The cause of the trouble was finally decided to be the electrostatic repulsion of particles away from the center line of mass of the test cylinder with concentration at the side walls. The method was abandoned as unreliable, and rather definitely disposed of sedimentation methods depending on a collection of particles in fractional areas of containers.

The writer feels that a more elaborate method of utilizing the changing suspension densities offers a solution of the problem. While the hydrometer method is unsuitable for exact laboratory work, its objections or faults can be met in an apparatus in which a rod of uniform cross-section is suspended to a fixed level in a sedimentation cylinder and instantaneous records of its apparent weight are found by its being hung from one arm of a sensitive balance. As the suspended matter in the liquid settles out, the apparent weight of the rod will increase. The time rate of change in weight may be obtained by noting the times at which a given weight is equalled, electrical

[28] *Soil Science*, v. 19.

[29] *Proceedings and Papers*, First International Congress of Soil Science, by Charles F. Shaw and E. V. Winterer, June 13–22, 1927, Vol. 1.

contacts on the balance recording the instant. A magazine can then release a known weight increment upon the opposing balance pan, the balance will swing back, and a new time interval will be recorded following sufficient change in density. The vertical travel may be kept less than a millimeter, by the use of stops under the pan or arms.

The proposed apparatus would avoid Fault (*a*), by having a constant subsiding range; Fault (*b*), by having a truly cylindrical body; and Fault (*c*), by not disturbing the rod after the test had started. Fault (*d*) would not exist, and Fault (*e*) could be avoided by Rubey's graphical, or Oden's mathematical, methods.

The falling range for subsiding particles will be the distance from the surface of the liquid to the lower end of the rod. This fact was established by the writer during the 1924 work. It was found that a cup submerged in clear water, carrying suspended material, registered its computed weight when the bottom was impervious, but registered only the weight of the matter resting on the bottom of the cup, when an aperture was opened therein. The change in weight was immediate and indicated that that weight was transmitted as a pressure due to frictional effects on the falling particles and not as an increased suspension density which would have affected the weight of the cup until slow diffusion through the aperture had taken place. Hence, the suspended rod will be acted upon vertically upward by all the displaced suspended matter above its lower end and by none of the matter that has passed this horizon.

An automatic data-recording device is desirable to avoid the need of close personal attention during 24-hour runs. The weight increments should be no larger than is necessary to bring the balance back promptly. In the 1924 work, small steel ball-bearings weighing 0.05 gramme were found to be uniform in weight. They can be discharged upon the balance pan from a magazine actuated by the same electric current that marks the swing of the balance.

The writer regrets that he has not had an opportunity to experiment with this apparatus. Inasmuch as he believes that other devices have failed to give much more than approximate results, this is suggested as a possibility, with the hope that some dependable analytic procedure can be developed. If such procedure were found, it would then be possible to compare less elaborate analytical devices, such as Bouyoucos' hydrometer, to determine the extent of their reliability and establish calibration corrections thereto.

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DISCUSSIONS

CONSTRUCTION WORK ON A FEDERAL RECLAMATION PROJECT

Discussion

BY MESSRS. EDWARD W. BUSH, AND ORRIN H. PILKEY

EDWARD W. BUSH,[7] M. AM. SOC. C. E. (by letter).[7a]—Near the end of this interesting paper, under "Construction Management," the author when discussing construction costs makes the statement that:

"It is difficult to understand why one contractor should receive 100% more payment per unit than another operating on the same work and under the same conditions. It indicates that the competitive system of bidding is at fault."

The writer offers the suggestion that probably the greater part of these discrepancies were caused by the contractors unbalancing their bid items either by increasing the prices on the items completed during the early part of the work—and thus obtaining money needed to finance the job—or by increasing the amounts bid on items which it was thought might be increased in quantity and, at the same time, decreasing the prices on other items which might be decreased in quantity. Competitive bidding is not to blame for the discrepancies mentioned by the author and, in most cases, these discrepancies can be prevented if the engineer, in the instructions to bidders, includes a condition stating that grossly unbalanced bids will not be given consideration when making the award.

It is thought that most engineers expect a certain amount of unbalancing unless they include in the set-up items like installation of equipment, organization expense, over-head, pumping, coffer-dams, etc., which often are real items appearing in the cost estimate of the bidder and which he naturally desires to load on to the early items if this is possible. It is not unusual to find items like foundation excavation running from $6 to $10 per cu. yd., provided a coffer-dam not elsewhere paid for must be built before the "pay" yardage work is started. In general, it is to be expected that, whenever the

NOTE.—The paper by Morris Mason, Jun. Am. Soc. C. E., was published in October, 1931, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: December, 1931, by John Sanford Peck, Assoc. M. Am. Soc. C. E.

[7] Engr., Aetna Casualty & Surety Co., Hartford, Conn.

[7a] Received by the Secretary December 2, 1931.

engineer has carefully analyzed the work to be done and has set up plenty of items so that each part can be bid separately, rather close bids will be received from the really experienced contenders for the job.

The defense against the gross unbalancing of items that might be increased or diminished in quantity is found in making such thorough preliminary investigations of the site or in working out the details of the design so that little doubt exists as to the quantities that will be required.

Some rather disturbing results may arise from grossly unbalanced bids on a unit price contract. The contractor may accumulate a considerable amount of early profits and expend them on some other jobs being performed at a loss; he may install equipment or an organization on other subsequent contracts; or, he may send his family to Europe, or otherwise expend the profits, and then lack sufficient funds with which to complete the contract on which the unbalancing occurred. Another interesting fact is that during the first part of the work the owner may be holding his own money as the retained percentage instead of any sum that has been earned by the contractor.

Even if the bid items are unbalanced a moderate sum, it is the total price that interests the owner and is considered when making the award; and if the case has been carefully prepared for bidding the totals coming from the experienced bidders will generally be found to compare favorably with one another and only vary because no two persons are apt to think alike on the probable cost or the amount of profit to be added. The average bid seldom has the importance frequently given to it, because there are generally a number of high bidders who only bid because they were asked, or they are bidders who lack an available plant or organization, or possibly a seasoned experience in doing similar work; hence they do not possess reliable cost data and, therefore, bid high. Some high bidders are supplied with enough current work to keep them fairly well occupied and only desire another contract if it can be secured at a favorable price.

Probably in most cases the real contenders for the contract are to be found in the lower third or half of the bids, provided those offering these bids are experienced contractors on the class of work bid upon. Complementary bids are generally high and many of these are offered merely because the engineer or architect sent the plans to the bidder. Others bid merely to see their names among those bidding, thinking that a certain prestige is obtained if the job happens to be a large one. Bankers at times will extend rather large credits for certified checks when it is known that the bidders will put in such high prices that the jobs will not be obtained except at very attractive figures. It is not worth while to give too much consideration to the high bidders if a fairly large number of bids are offered, but when judging the lowest bids before making an award it is desirable to give full consideration to the bidder's experience, financial strength, available plant, and organization; also, whether the new work taken in connection with the other work on hand will be inside the volume which the contractor is considered competent to handle and finance.

ORRIN H. PILKEY,[8] JUN. AM. SOC. C. E. (by letter).[8a]—This is a well-written paper on a Federal Reclamation project, and Mr. Mason is to be congratulated on the thoroughness with which he has treated his subject.

It is interesting to note that actual construction was begun on some high-line canals before the U. S. Bureau of Reclamation took part, but that this early attempt resulted in failure because of lack of funds. This is an example of the value of the Federal Bureau. With its greater financial backing it is able to undertake and complete desirable projects which otherwise would not be attempted, or, if attempted, would result in failure.

One of the features of the Kittitas Project worth mentioning is the use of safeguards of several types in the main canal to prevent accidents, especially drowning. Just above each siphon or tunnel intake is a creosoted timber float fastened to a 1-in. guide rope. If a person is being swept down with the current he might be able to catch hold of this timber and hang on to it, thus saving himself from being carried through the siphon or tunnel. If he missed the timber he would scarcely be alive by the time he reached the outlet.

In addition to the timber there are metal ladders placed on both sides of the canal at the siphon entrances. Other ladders are spaced about 250 ft. apart on alternate sides of the lined canal. Their purpose is to provide egress from the canal itself. The side slope of 1 on $1\frac{1}{4}$ of the concrete lining is so steep that it is almost impossible for a person to climb out when the concrete is wet and slippery.

Timber fences of heavy construction border the edges of the canal at both the inlet and outlet ends of the siphons, and metal railings built of angle irons are placed around the operating platforms. The bridges crossing the canal also have angle-iron railings.

The author has given figures which he states indicate that the competitive system of bidding is at fault. It is certainly interesting to note the great variations in unit prices paid to different contractors operating on the same work and under the same conditions. Some of the bids must have been extremely unbalanced. It is admitted that competitive bidding leaves much to be desired, but other methods of bidding on public work are likely to be far more unsatisfactory. The writer would like to inquire what the author recommends to alleviate the faults of the competitive system.

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[8] Care, Waddell & Hardesty, New York, N. Y.

[8a] Received by the Secretary December 7, 1931.

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DISCUSSIONS

WESTERN HIGHWAY PRACTICE, WITH SPECIAL REFERENCE TO CALIFORNIA

Discussion

BY MESSRS. J. S. BRIGHT, AND R. D. RADER

J. S. BRIGHT,[3] M. AM. SOC. C. E. (by letter).[3a]—The author has presented some interesting developments in highway construction practice. The items of highway finance, stream erosion, floods, cloudbursts, sea action, clearing in advance of construction, and construction through marshes, are always interesting due to their general application.

Finance.—In the eleven Western States, as indicated by Mr. Pope, finance is a paramount question. During the 1928 season the Western States reported total funds for State highway purposes aggregating $91 500 000. The sources of this sum were: Motor fuel tax, $40 000 000; plate tax, $18 000 000; miscellaneous sources, $18 500 000; and Federal aid, $15 000 000.

These figures show that the Federal Government is contributing approximately 16% of the cost of State road construction in this area; or, taking it in the reverse order, the State systems are being united into a Federal system at a cost to the National Government of 16% of the programs. Furthermore, the State road users are paying about 76% of the State's share, through licenses and fuel taxes, leaving only 24% for other State interests to contribute.

With such a large contribution by the road users, it is apparent that road programming should give first consideration to the serving of traffic. A well-planned budget is paramount for good administration. It should cover a series of years, letting the improvements advance in an orderly manner. Service to the public should demand that budgeting prevent dead-end projects, but traffic should be served during construction operations by betterments on existing roads and "stage" construction.

NOTE.—The paper by C. S. Pope, M. Am. Soc. C. E., was presented at the meeting of the Highway Division, Sacramento, Calif., April 24, 1930, and published in November, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[3] Constr. Engr., U. S. Bureau of Public Roads, San Francisco, Calif.

[3a] Received by the Secretary November 30, 1931.

There is nothing fiscally unsound in "stage" construction if it is practiced with the major purpose of serving traffic. This "stage" construction may involve narrow surfaces, pioneer roads with short stretches of steep grades, concrete fords in lieu of expensive structures, and temporary oil-processed surfaces on old county roads. Such oil-processed material can be salvaged readily and transported to the final location. The oil-processing of base courses for serving traffic is also an economical expenditure where mileage and short seasons are governing factors.

Erosion.—Two features of construction may cause erosion: The destruction of forest cover and the filling of watercourses with large quantities of waste excavation.

These conditions are most serious where flashy storms occur on water-sheds adjacent to highly industrialized areas. Nevertheless, the borders of all California roads have a real value for their scenic attractiveness. This attractiveness should be capitalized and not disregarded in construction operations.

The U. S. Bureau of Public Roads, during the 1931 season, has been adopting special features for the protection of forest cover on highway operations in the National Parks. This was brought about at the insistence of the National Park Service, which is to be commended for providing the necessary funds for carrying it to a successful conclusion.

This protection has been accomplished by carefully developed plans, and the placing of necessary safeguards in the specifications. The landscape engineers, on the preliminary locations, have designated the features to be preserved. These features have been carefully studied during the location, and measures then adopted for necessary protection. The best protection measures have been full bench sections, balancing fills, tunnels, hand-laid embankment, concrete cribs, and retaining walls.

Specifications for construction in the National Parks have provided clearing by stages, leaving a screen of timber on the lower side until after the heavy shooting has been finished. Prior approval is required for the plan of drilling and loading. It is contemplated that the engineer will approve this plan on its successful demonstration over a small area. The specifications give the engineer, as a last resort, authority to limit the spacing of holes to 2-ft. centers, and a minimum charge of $\frac{1}{2}$ lb. of 40% dynamite per cu. yd. of material. Mats may also be required as a last resort. Penalties consist of gathering the scattered material, the removal and trimming of injured trees, or, in extreme cases, closing down the work. Six contracts under these specifications show an increase of 57% in cost for 25% of the total excavation. The additional total cost, therefore, has been nominal.

The preventive measures adopted by contractors have been mostly in blasting. Quarrying the rock at the end faces of the cuts accomplishes the desired results, but is not adapted to an efficient program for drilling. Better results have been obtained by making an initial cut along the line of the upper ditch. This permits subsequently blasted material to fall inward, rather than over the slopes. The use of narrow pioneer fills near the lower slope stakes has given good results. This method provides room on the upper side for dumping large rock fragments.

One contractor uses half-round corrugated metal culvert sections for protecting the trunks of trees. In addition, the use of heavy woven-wire, guard fencing, strung between the first line of trees, has been proposed for checking rolling rocks.

Hand-laid embankments have proved satisfactory and have involved a cost of $1.40 to $2.00 per cu. yd. more than the cost of rock excavation. Reinforced concrete cribs are adapted to the same purpose, and have been costing from $2.00 to $6.00 per sq. ft. of exposed face.

In cliffs, the tunnel is an ideal design to prevent destruction of forest cover below, and also to prevent waste material from falling. The West has become "tunnel minded"; although many projects have been designed with short tunnels, the tendency seems to be for greater lengths. No trouble has been experienced from engine gases on lengths of about 1 500 ft. The ventilation problem in the Western canyons is quite different from that encountered in river tunnels, where the dipping grades generally produce pockets. Tunneling has proved economical, the prices varying from $55 to $100 per ft. for a 23-ft. unlined roadway section, and from $80 to $130 per lin. ft. for lined tunnels.

Advance Clearing.—The author has opened the question of clearing in advance of construction. This practice accomplishes several desirable objects, among which, first and foremost, is the desirability of burning when the conditions of fire hazard are the least. It also allows the slope stakes to be placed well in advance of construction operations and provides sunlight for the early melting of snows with resulting lengthening of the construction season.

Construction Through Marshes.—The problem of construction through muskeg or marsh is a live question in Alaska, and is there solved by ditching and corduroy road building. Logs for corduroy construction should be from 4 to 8 in. in diameter, and should provide a ridged platform. A 6-in. seal of fine roadside material is placed over the corduroy before the gravel sub-base is applied. The seal excludes the air, which causes decomposition. Examined after ten years corduroy roads show no signs of decay where such a seal has been applied, but where gravel has been used directly on the logs the roadway has broken badly in a few years.

A planking to carry construction traffic, even over the completed gravel surface, has been found necessary. This affords time for the adjustment to take place uniformly, while the ditches are drawing down seepage to a constant flow. The ditches should be at least 10 ft. beyond the line of the shoulders, and of generous depth. Through some areas it has been found advisable to construct the ditches well in advance. In general, planking will permit traffic immediately upon the roadway.

Conclusions.—A financial budget for highway construction is absolutely necessary, but the questions of what methods to adopt on problems of erosion, clearing, or wet foundations, are not so well defined. These problems must be considered separately, under existing physical and financial conditions. It is certain, however, that where scientific principles are applied with patience the correct solutions will appear.

R. D. RADER,[4] M. AM. SOC. C. E. (by letter).[4a]—This excellent paper covers a number of the problems with which highway engineers are confronted, some of them being peculiar to the Western States.

The eastern half of Montana is a plateau from 3 000 to 4 000 ft. above sea level and is especially subject to so-called "cloudbursts." These occur during the hot summer months, and often cover only a few square miles, but as much as 2 in. of rainfall may occur in less than an hour. Mr. Pope states that it is the opinion of many engineers that cloudbursts occur in approximately the same areas within a reasonable range of time. Eleven years of observation of this phenomenon in Montana leads the writer to believe that there is no possibility of anticipating where cloudbursts will occur except in a general way. Of course, certain large areas are more subject to these torrential rains than others, but it is only occasionally that they are found to cover the same acreage, or that the run-off crosses the highways at exactly the same place. Some of the alluvial "fans" or "cones" spread out several miles in width and unless the highway is located along the foothills so as to bridge across the main channels where they are well defined, it is difficult to control the flow so as to confine it to bridges of reasonable length.

In Montana, such an elaborate system of ditches and dikes as that described by Mr. Pope has not been attempted. The first rush of water,

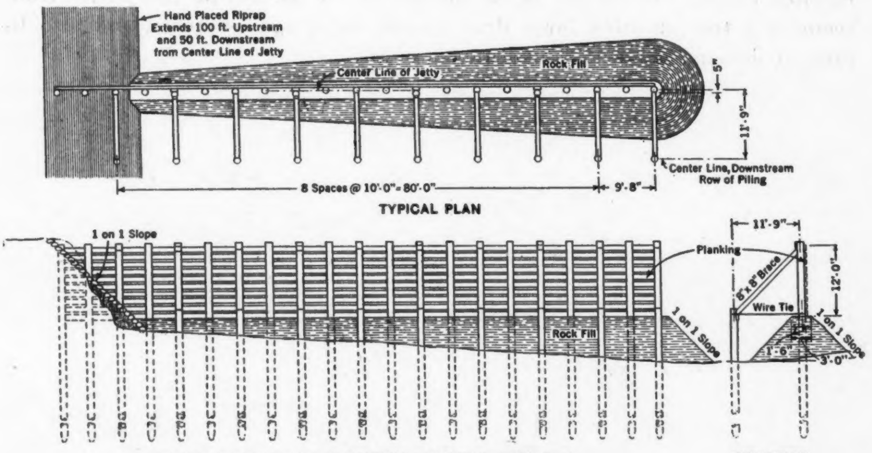

FIG. 15.—PILE AND PLANK JETTIES ALONG THE BIG HORN RIVER, NEAR CUSTER, MONT.

sand, boulders, and débris will fill any ordinary intercepting ditch, and these ditches have not served the desired purpose of directing the flow under the bridges. However, such flow is usually of short duration, and passes over the highway in a broad flat sheet which does little damage, except the scouring and gullying of the shoulder on the down-stream side of the highway.

[4] State Highway Engr., Helena, Mont.

[4a] Received by the Secretary December 4, 1931.

The cost of repairing this damage is less than the interest charge on an adequate system of ditches and bridges. The method followed in Montana, therefore, is to build bridges over the natural channels, designed so as to carry the normal run-off from the usual rains or melting snow. No attempt is made to design and build bridges for the cloudbursts which may not occur more than once in a long period of years. Occasionally, the grade line of the highway is depressed for a few stations so as to allow exceptional floods to flow over, and, where the grade is well compacted, the loss of embankment is found to be small, even if there is no protecting wall or rip-rap.

For bank protection of rivers flowing through soil subject to scour, the method of driving double rows of piling parallel to the stream has been used successfully in Montana. Brush and rock are placed between the piling, and are retained by planking or wire mesh attached to it. In such cases, if possible, the piling is driven deep enough to be below the probability of scour. Pile jetties, also, are sometimes used. Fig. 15 shows a layout of pile and plank jetties installed to protect a scouring bank on the up-stream side of a bridge over the Big Horn River, near Custer, Mont. This river carries a large volume of silt in its flood stages, and it is expected that the partly planked, piling jetties will retard the current at flood stage so as to cause the deposition of this silt and thus restore the bank. If this occurs, willows will be planted between the jetties in order further to protect the newly restored bank. The rip-rap is for the purpose of protecting the jetties from scour if a tree or other large drift should lodge against them, and also to prevent damage from floating ice.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer*, and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibilty. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that errors in the record be pointed out and a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from January 15, 1932.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

LIST OF APPLICANTS.

Names and Addresses of Applicants for Admission and for Transfer on this List, Arranged Alphabetically.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

The number in the center above each record indicates the serial number of the applicant for the current year, and that at the left the district in which he resides.

The abbreviations in Italics represent respectively, *TT*, Total Time; *SP*, Sub-Professional Work; *P*, Professional Work; *RC*, Responsible Charge; *D*, Design. The figure for Total Time is determined by adding one-half the time spent in Sub-Professional Work to the time spent in Professional Work. The figures showing the amount of time spent in Responsible Charge and on Design are the estimate of the Applicant. The allowance of four years for graduation or of one-half of a year for each academic year successfully completed in an engineering college without graduation is included in Total Time and Professional Work.

FOR ADMISSION

1

(15) ADAMS, WALTER KELSEY, 817 West Magnolia Ave., San Antonio, Tex. (Age 53. Born Oneonta, N. Y.) 1903 B.S. in Civ. Eng., Univ. of Wis. *TT 4: P 4.*—June 1903 to March 1907 Rodman, Instrumentman, etc., until March 1905 with Pennsylvania Lines West, then with Chicago & Northwestern Ry., also Pile Bridge Inspector for latter. *TT 1.9: SP 1.9.*—March to Dec. 1907 Draftsman, until Sept. 1907 with Northwestern Elevated R.R., Chicago, on structural designing, then with Lassig Plant, American Bridge Co., on structural detailing. *TT 0.3: SP 0.3*—Jan. 1908 to May 1914 with National Rys. of Mexico, until March 1910 as Asst. Engr., Dist. Engr., M. of W. Dept., then Res. Engr., Constr. Dept., on Rio Grande Bridge, Laredo, Tex., and after June 1911 Bridge Engr., M. of W. Dept., on general repairs and reconstruction of track and bridges. *TT 5.6: SP 0.9: P 4.7: RC 3.*—June 1914 to Aug. 1921 with U. S. War Dept. until Aug. 1917 and after Nov. 1919 as Jun. Engr., tabulating railway data and formulating plans for organization and operation of military railways in Mexico, from Aug. 1917 to Nov. 1919 Capt., Engrs., U. S. Army, 16 months with A. E. F. on maintenance of tracks, in yards of Gen. Intermediate Storage Depot at Gievres, France, and in charge of construction of rest camp and hospital at Southampton, England. *TT 6.4: SP 0.6: P 5.8: RC 2.2.*—Aug. 1921 to Feb. 1922 not on engineering work.—Feb. to Sept. 1922 with City Engr.'s. Dept., San Antonio, Tex., in charge of reconstruction of South Alamo St. Bridge over San Antonio River, and Chf. of Party on paving and surveys. *TT 0.5: SP 0.2: P 0.3: RC 0.3.*—Sept. 1922 to Sept. 1924 Asst. Engr., San Antonio & Aransas Pass R.R., in charge of design and construction of substructure for Brazos River Bridge at Wallis, Tex. *TT 2: P 2: RC 1: D 1.*—Sept. 1924 to Sept. 1926 Chf. of Party, Flood Prevention Dept., City of San Antonio, on surveys and on location and construction of park roads in connection with Olmos Dam. *TT 2: P 2.*—Sept. 1926 to Nov. 1928 Asst. Engr., Missouri-Kansas-Texas Ry., being Transitman and Chf. of Party on surveys. *TT 2.1: P 2.1: RC 0.4.*—Jan. 1929 to Sept. 1931 Chf. Engr., Montgomery & Co. (subsidiary of United Fruit Co.), Ixtapa, Jalisco, Mexico, in charge of design and construction of abutments for and erection of truss (450 ft.) bridge, with pile trestle approach, crossing Mascota River, and gravity irrigation system for plantation (about 3,000 acres); preliminary surveys for gravity and pumping systems to irrigate 7 000 acres; built light railway (16 km.) with necessary sidings and yards and prepared plans and estimates for extension, on preliminary plans and estimates and studies for proposed construction of loading pier. *TT 2.8: P 2.8: RC 2.8: D 2.8.*—*TT 27.6: SP 3.9: P 23.7: RC 9.7: D 3.8.* Refers to E. P. Arneson, S. F. Crecelius, E. L. Daley, G. E. Edgerton, W. S. Goodman, G. J. Kennedy, R. P. Parker, E. W. Robinson, H. M. Taylor, A. Y. Walton.

2

(14) ATKINSON, GEORGE LEE, 1908 Wolfe St., Little Rock, Ark. (Age 27. Born Galveston, Tex.) April 1926 to March 1927 Asst. Engr. on Street Improvement District work, until June 1926 for Frank Pritchett, then for W. Terry Field, both of Little Rock, Ark. *TT 0.9: P 0.9: RC 0.9.*—March 1927 to April 1928 with Arkansas State Highway Dept., until July 1927 as Draftsman and Designer, then Designer and Asst. Res. Engr. on construction, Dist. No. 6, Little Rock. *TT 1.1: P 1.1: RC 0.4: D 0.6.*—April 1928 to June 1931 Structural Draftsman, Detailer and Asst. on bridge surveys, Arkansas Bridge Dept. *TT 3.2: P 3.2: RC 3.2.*—*TT 5.2: P 5.2: RC 4.5: D 0.6* Refers to P. Andersen, C. S. Christian, N. B. Garver, A. M. Lund, D. A. MacCrea, W. A. Poe.

3

(4) BAILEY, JULIAN GEORGE, 266 South Fifty-eighth St., Philadelphia, Pa. (Age 53. Born Chicora, Pa.) June 1899 to July 1900 Rodman and Chainman on railroad work. *TT 0.5: SP 0.5.*—July 1900 to Aug. 1907 with Pennsylvania R. R., until 1905 as Transitman (Chf. of Party) and Chf. Masonry Inspector on elevation of Monongahela Div., freight-line construction, etc., then Asst. Engr. on preliminary survey, location, etc., for low-grade freight line, Glen Loch to Philadelphia, other surveys, etc. *TT 4.5: SP 2.5: P 2: RC 3: D 1.*—Aug. 1907 to Sept. 1909 Civ. Engr. and Surveyor, Coatesville, Pa., supervised construction of plant for Coatesville Foundry and Machine Co., surveyed and designed plant for Green Lane Trap Rock and Ice Co., including dam, ice houses and quarry plant, and other engineering work. *TT 2.1: P 2.1: RC 2.1: D 1.*—Sept. 1909 to July 1911 Field Engr., Inter-State Eng. & Supply Co., Philadelphia, in charge of construction of electric power plant for Metropolitan Elec. Co., Reading. *TT 1.8: P 1.8: RC 1.8.*—March 1912 to Oct. 1917 Asst. Engr. and Dist. Engr., Bureau of Highways, Philadelphia, in charge of construction of boulevards and highways. *TT 5.6: P 5.6: RC 5.6: D 2.*—March 1920 to Jan. 1921 Civ. Engr., Du Pont Eng. Co., Wilmington, Del., being Cons. Engr. on reorganization work, studying construction operations and costs and advising changes in methods, etc., on various projects, such as new plants for Cadillac Automobile Co., Detroit, Mich., Buick Automobile Co., Flint, and General Motors Truck Co. and Oakland Automobile Co., Pontiac, Brown Life Chapin Co., Syracuse, N. Y., etc. *TT 0.7: P 0.7: RC 0.7.*—Oct. 1917 to March 1920 and April 1923 to May 1930 Field Engr., Day and Zimmermann, Inc., Philadelphia, until March 1920 supervising construction of steel plant for Erie (Pa.) Forge and Steel Co., surveying, and laying out town site for U. S. Shipping Board, south of Gloucester, N. J., and construction of Philadelphia Quartermaster's terminal for U. S. Army (two large 3-story reinforced concrete warehouse piers on wooden piles, hydraulic dredging of Delaware River, pier slips and railroad connections, and after April 1923 making investigations and preliminary reports on development of proposed hydro-electric projects, principally topographic surveys in United States and Canada. *TT 9.5: P. 9.5: RC 9.5: D 6.5.*—May 1930 to date Constr. Engr. in charge of Constr. Div., Bureau of Eng., Dept. of Public Works, Philadelphia, supervising building of bridges, sewers, sewage-disposal and miscellaneous structures, etc. *TT 1.5: P 1.5: RC 1.5.*—*TT 26.2: SP 3: P 23.2: RC 23.2: D 10.5.* Refers to W. C. Bailey, W. H. Connell, J. H. Neeson, C. Penrose, G. F. Rowell, E. R. Schofield, F. G. Schworm, E. B. Temple.

4

(4) BARRICK, MERVIN JOSEPH, 1603 Junction St. South, Williamsport, Pa. (Age 43. Born Steelton, Pa.) Prof. Engr., State of Pennsylvania. 1911 B. S. in San. Eng., Pa. State Coll. *TT 4: P 4.*—July 1911 to April 1912 Draftsman, Pennsylvania Steel Co., Steelton, Pa. *TT 0.4: SP 0.4.*—May to Oct. 1912 Inspector, New York Agricultural and Industrial School, Industry, N. Y. *TT 0.2: SP 0.2.*—Nov. 1912 to April 1913 Rodman, Wabash Pittsburgh Terminal Ry. Co. *TT 0.2: SP 0.2.*—May to Dec. 1913 Draftsman and Inspector with Chester & Fleming, Pittsburgh, on water-works. *TT 0.4: SP 0.4.*—Feb. 1914 to July 1915 and Nov. 1915 to June 1916 Draftsman, etc., on water-works and sewers, successively with L. E. Chapin, Pittsburgh, and Farley Gannett, Harrisburg, Pa. *TT 1.4: SP 0.9: P 0.5: D 0.3.*—July to Dec. 1916 and Feb. to Aug. 1917 Foreman on filter construction, successively with Pitt Constr. Co. and Pihl & Miller, both of Pittsburgh, latter period in charge. *TT 0.9: SP 0.2: P 0.7: RC 0.7.*—Sept. 1917 to July 1919 with U. S. Army, until June 1918 as Private, Corporal and Sergeant, Infantry, Utilities and Eng., being in charge of filter operation at Camp Meade, Md. (5 months), and after July 1918 2d Lieut., 311th Engrs., 86th Div., in charge of detached platoon on road work, Montpont, France (3 months). *TT 1.3: SP 0.5: P 0.8: RC 0.8.*—Aug. to Dec. 1919 Draftsman and Inspector, Americal City Eng. Co., Pittsburgh, on town planning. *TT 0.2: SP 0.2: D 0.1.*—Jan. to Sept. 1ε20 Draftsman and Asst. with Douglass & McKnight,

Borough Engrs., Pittsburgh. *TT 0.6: SP 0.2: P 0.4: RC 0.1: D 0.3.*—Oct. 1920 to date with Pennsylvania Dept. of Health, Harrisburg, until Dec. 1925 inspecting waterworks operation, 9 months as Eng. Asst. and 4½ years Asst. Engr., and since Jan. 1926 Dist. Engr., Williamsport Office. *TT 10: SP 1.3: P 8.7: RC 8.7: D 3.*—*TT 19.6: SP 4.5: P 15.1: RC 10.3: D 3.9.* Refers to D. E. Davis, A. E. Farrington, J. R. Hoffert, M. C. Krause, H. E. Moses, W. L. Stevenson, E. D. Walker.

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(3) **BENTLEY, ELROY WILLIS,** 79 Park St., Glens Falls, N. Y. (Age 48. Born Glens Falls, N. Y.) Graduated, International Correspondence Schools.—1902 to March 1930 with New York State, Canal and Highway Depts., until 1911 (winters) putting side wall and concrete bottom in canal, repairing lock gates, looking after stone quarry, being Mixer, Foreman and Timekeeper; April 1911 to July 1921 Highway Inspector, July 1921 to Jan. 1926 Jun. Asst. Engr., and after Jan. 1926 Asst. Engr., Grade 1 and field and office work, during 1913 in charge of contract and since April 1914 Engr. in charge of Delaware County, including construction of roads, bridges, railroad elimination and all maintenance. *TT 17.5: SP 1.5: P 16: RC 16*—1930 to date in private practice as Surveyor, real estate, appraisals and insurance, at Glens Falls, made floor plans for remodelling, etc.—*TT 19: SP 3: P 16: RC 16.* Refers to C. T. Fisher, R. B. Hoadley, Jr. A. T. Paine, W. C. Ruland, E. E. Stickney, J. H. Thomas.

6

(1) **BRIGHAM, RICHARD WINANS,** 885 East Thirty-eighth St., Brooklyn, N. Y. (Age 28. Born Binghamton, N. Y.) 1927 B. S. in C. E., Union Coll. *TT 4: P 4.*—June 1927 to date with New York Telephone Co., until Sept. 1928 as Eng. Asst., Constr. Dept., preparing routine orders and since Sept. 1928 Outside Plant Engr., supervising engineering work and preparing specific estimates. *TT 4.5: P 4.5: RC 3.3.*—*TT 8.5: P 8.5: RC 3.3.* Refers to A deH. Hoadley, H. Miller, E. H. Prentice, H. A. Schauffler, W. C. Taylor.

7

(1) **BROBERG, FREY LORENTS,** 50 West Front St., Red Bank, N. J. (Age 45. Born Malmo, Sweden.) 1906 graduated, Malmo Tech. Coll.—Aug. 1906 to April 1912 Draftsman, Designer and Engr., Hayward Co., on design of cranes, excavating and dredging, coal and sand-handling and traveling and floating hoisting machinery. *TT 3.8: SP 1.8: P 2: RC 1: D 1.*—May 1912 to July 1914 Designing and Constr. Engr., Nitrate Agencies, Iquaque, Chile, remodeling several saltpeter refineries, and design and construction of Oficina Paposo. *TT 2.2: P 2.2: RC 2.2: D 1.*—July 1914 to June 1916 in charge of construction equipment of Constr. Dept. of Swedish Govt. R. R., acted in consulting capacity on construction methods, etc.—June 1916 to May 1917 Constr. Engr., C. G. Lianossof, Petrograd, on design and construction of and Operating Engr. for munitions manufacturing plant. *TT 0.7: P 0.7: RC 0.7: D 0.3.*—June 1917 to April 1918 Draftsman, Chile Exploration Co., on design of machine for casting copper ingots. *TT 0.8: P 0.8: D 0.8.*—April 1918 to Jan. 1919 Engr., T. W. Price Co., on design and construction of steel foundry in East Chicago, Ind., various industrial plant designs and supervision of construction. *TT 0.6: SP 0.1: P 0.5: RC 0.3: D 0.2.*—Jan. 1919 to April 1923 Constr. and Mech. Engr., Compania Minera, Choco-Pacifico, Colombia, on design and construction of placer mining dredge, river surveys in Choco, investigations and reports. *TT 3.8: SP 0.5: P 3.3: RC 2: D 1.3.*—1923 to 1926 travelling; at times acted as Cons. Engr. regarding construction equipment; supervised design of gate-operating mechanism for hydro-electric power plants.—Dec. 1926 to Sept. 1927 Mech. Engr., Raymond Concrete Pile Co., on design of special equipment for construction of concrete bridge over Lake Pontchartrain, La., and equipment for Buena Ventura and Maracaibo Harbors. *TT 0.8: P 0.8: RC 0.1: D 0.7.*—Jan. to May 1928 Mech. Engr., Frederic Snare Corporation, on design of special equipment for construction of Port of Callao, Peru. *TT 0.4: P 0.4: D 0.4.*—May to Nov. 1928 Valuation Asst., Coverdale & Colpitts, on valuation of structural steel, concrete construction, development of costs for both overhead and tunnel, construction for Interboro Rapid Transit R. R. *TT 0.4: P 0.4: RC 0.4.*—Dec. 1928 to Oct. 1929 Asst. Port Engr., Ulen & Co., Persia, on survey and mapping Jerrachi River to Persian Gulf, hydrographic survey of Persian Gulf from Bu Sif north, assembling survey in marine charts. *TT 0.8: SP 0.1: P 0.7: RC 0.7.*—Feb. 1930 to July 1931 Plant Designer, Anglo Chilean Nitrate Corporation, on designs of screening, sampling, crushing, reject-disposal, and dust-collecting and dust-disposal plants. *TT 1.4: SP 0.1: P 1.3: D 1.3.*—*TT 15.7: SP 2.6: P 13.1: RC 7.4: D 7.* Refers to R. R. Ellis, Jr., H. H. Friendly, L. A. Jenny, J. I. Leonard, T. G. Redington, C. E. Seage, H. A. Vanderbeek, W. D. Volk.

8

(1) CHRISTENSEN, JOHN JOSEPH, 1143 Clove Rd., West New Brighton, N. Y. (Age 24. Born Perth Amboy, N. J.) 1930 B. S. in E., Manhattan Coll. *TT 4: P 4.*—Summer 1929 and Feb. 1931 to date Eng. Asst., Board of Transportation, City of New York, being Chainman, Rodman, Inspector of construction, Draftsman and Transitman. *TT 0.4: SP 0.4.*—*TT 4.4: SP 0.4: P 4.* Refers to J. J. Costa, J. H. Myers.

9

(14) CLARKE, TIMOTHY JOHN, 1441 Peabody Ave., Memphis, Tenn. (Age 48. Born Clinton, Iowa.) Sept. 1904 to Sept. 1907 student, Univ. of Iowa. *TT 1.5: P 1.5.*—Feb. 1903 to Sept. 1904 with Iowa Eng. Co., Clinton, Iowa, 3 months as Rodman, 6 months Inspector, and 8 months on paving, sewer grades and surveys. *TT 0.7: SP 0.7.*—Sept. 1907 to Feb. 1908 Asst. Locating Engr. on railroad survey, Clinton to Dubuque, Iowa, under R. C. Hart. *TT 0.4: P 0.4.*—June 1908 to date Engr.-in-Chg., Clarke Bros. Constr. Co., Memphis, Tenn., on bridges, sewers and paving ($100 000), designed and built two drag-line excavators ($20 000), drainage work (tile and open, $900 000), installed a Diesel electric drag line and an electric tower excavator and power house, and on Mississippi flood-control levee work, consisting of Bird Point New Madrid spillway and drainage and Davis Lake Levee, Ark. (total $655 000). *TT 20: P 20: RC 19: D 1.*—*TT 22.6: SP 0.7: P 21.9: RC 19: D 1.* Refers to L. T. Berthe, L. L. Hidinger, J. R. Rhyne, W. A. Vaught, H. A. Wiersema.

10

(14) CUMMINS, WILLIAM FRANKLIN, Jr., 1513 Cherry St., Vicksburg, Miss. (Age 24. Born Yazoo City, Miss.) 1930 B. S. in Mech. Eng., Ga. School Tech. *TT 4: P 4.*—June 1930 to date with U. S. Engr. Office, Vicksburg, Miss., 1 year, as Inspector and 6 months Jun. Engr., on studies, calculations and reports on power problems, flood control and navigation, preliminary structural designs and estimates. *TT 1.5: P 1.5: RC 0.5.*—*TT 5.5: P 5.5: RC 0.5.* Refers to O. G. Baxter, W. M. Borgwardt, F. G. Christian, G. R. Clemens, K. R. Young.

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(15) DAVIS, ARNOLD MARION, 406 Triad Bldg., Baton Rouge, La. (Age 29. Born Newberry, S. C.) 1½ years student, Georgia School of Technology. *TT 0.5: P 0.5.*—June 1923 to June 1926 Associate of Hickman-Davis Realty & Eng. Co., Miami, Fla., on surveys, sub-division street work, etc. *TT 3: P 3: RC 3.*—June 1926 to June 1929 Res. Engr. on asphalt projects in Harrison County, Miss. (1½ years) and for South Carolina Highway Comm. (1½ years) also two sand-clay road projects for latter, being in charge of work. *TT 3: P 3: RC 3.*—June 1929 to date Res. Engr. in charge, until Sept. 1931 of asphalt work in Dist. No. 1,, Arkansas State Highway Dept. and since Sept. 1931 of asphalt projects for Louisiana Highway Comm., Baton Rouge, La. *TT 2.4: P 2.4: RC 2.4: D 2.2.*—*TT 8.9: P 8.9: RC 8.4: D 2.2.* Refers to R. L. Bannerman, S. A. Brady, C. H. Christian, C. S. Constant, E. M. Davis, A. C. Galt.

12

(1) DE BANG, HENNING (formerly Kornerup-Bang, Henning Samuel), 40 Sidney Pl., Brooklyn, N. Y. (Age 36. Born Viedbjerg, Denmark.) 1921 B. S. in C. E., Royal Pol. Coll., Copenhagen, Denmark. *TT 4: P 4.*—Dec. 1921 to May 1923 Asst. Engr. to Chf. City Engr., Esbjerg, Denmark, responsible field and office work. *TT 1.2: P 1.2: RC 1.2: D 0.6.*—April 1923 to April 1924 Asst. Engr., Royal Fire Brigade, Engr. Office, Biegdamsvej, Copenhagen, Denmark, designing and surveying. *TT 1: P 1: RC 0.5: D 0.5.*—April 1925 to Jan. 1926 Constr. Inspector, New York Steam Corporation, New York City, field inspection and supervision of steel and concrete construction. *TT 0.7: P 0.7: RC 0.7.*—Jan. 1926 to May 1927 Hydr. Designer and Checker, Elec. Bond & Share Co., New York City, on hydraulic, steam and electric power stations, turbine foundations, gravity dams, penstock design. *TT 1.3: P 1.3: RC 1.3: D 1.3.*—May 1927 to March 1928 Reinforced Concrete Designer, Ford, Bacon & Davis, Inc., New York City, on steam-electric power station, turbine foundations, retaining walls. *TT 0.9: P 0.9: RC 0.7: D 0.9.*—March to Dec. 1928 Structural Steel and Concrete Designer, Union Carbide & Carbon Corporation, New York City, chemical laboratories and compress stations, foundations, retaining walls, mill building design. *TT 0.7: P 0.7: RC 0.5: D 0.7.*—Jan. 1930 to Oct. 1931 Designer and Detailer, New York Central R. R. Co., New York City, on grade-crossing eliminations and express highway design and detailing structural steel and reinforced concrete. *TT 1.8: P 1.8: RC 1.8: D 1.8.*—April 1924 to April 1925, Dec. 1928 to Jan. 1930 and Oct. 1931 to date Engr. and Designer, International Cement Corporation,

New York City, designing steel and reinforced concrete machinery layouts on cement plants, heavy machinery foundations, structural steel mill building, silos, warehouses, steel tower, continuous reinforced slabs and beams in concrete. *TT 2.4: P 2.4: RC 1.9: D 2.4.*—*TT 14: P 14: RC 8.6: D 8.2.* Refers to E. M. Derby, T. B. Rights, Z. H. Sikes, H. T. Welty, E. E. Wissotsky.

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(15) DENT, JOSEPH BAKER, College Station, Tex. (Age 27. Born Foster Falls, Va.) 1926 B. S. in C. E., Va. Pol. Inst. 1931 M. S. in C. E., Agri. and Mech. Coll. of Tex. *TT 4: P 4.*—June 1926 to Sept. 1928 Rodman and Inspector, Norfolk & Western Ry. Co., field and office drafting. *TT 1.1: SP 1.1.*—Sept. 1928 to date Instructor, Eng. Drawing Dept., Agricultural and Mechanical Coll. of Texas, teaching engineering drawing and descriptive geometry. *TT 3.2: P 3.2.*—*TT 8.3: SP 1.1: P 7.2.* Refers to R. B. H. Begg, J. T. L. McNew, J. J. Richey.

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(11) DICKEY, WALTER LINNAEOUS, 5228 Eagle Rock Blvd., Los Angeles, Cal. (Age 25. Born Santa Clara, Cal.) 1931 B. S. in C. E., Cal. Inst. Tech. *TT 4: P 4.*— 1926 to 1927 and summers while student with J. V. McNeil Co., Contractors, running concrete mixer and placing concrete (7 months), placing reinforcing steel (4 months) and Rodman (1 month); summers general construction work. *TT 0.5: SP 0.5.*—*TT 4.5: SP 0.5: P 4.* Refers to F. J. Converse, B. A. Hill, R. R. Martel, W. W. Michael, F. Thomas.

15

(1) DI MICELI, BERNARD JAMES, 1133 Intervale Ave., New York City. (Age 24. Born New York City.) 1931 B. S. in Eng., Coll. of City of N. Y. *TT 4: P 4.*—Sept. 1926 to Sept. 1931 (evenings and summers) with Pamadica Constr. Corporation, New York City, as Estimator, Eng. Clerk, Timekeeper and Eng. Asst., making quantity and cost estimates for dwellings, garages and apartment houses (5 and 6 stories), checking time and materials on construction, assisting on layout of work, and responsible for layout and construction of an apartment house.—Sept. 1931 to date Jun. Asst. Civ. Engr., Grade 1, Long Island State Park Comm., on survey work, acting as Notekeeper, Rodman, Instrumentman, etc. *TT 0.2: SP 0.2.*—*TT 4.2: SP 0.2: P 4.* Refers to R. E. Goodwin, F. O. X. McLoughlin, J. C. Rathbun.

16

(7) DOUGLASS, ARTHUR SYLVESTER, 690 Burlingame Ave., Detroit, Mich. (Age 49. Born Plymouth, Mass.) 1908 student, Mass. Inst. Tech.—Summer 1908 Steel Detailer with Purdy & Henderson, Boston. *TT 0.3: P 0.3: D 0.3.*—1909 and 1910 Timekeeper, Rodman, etc. to Asst. Res. Engr., Colorado Power Co., Bear Tramp Dam, hard rock tunnel, hydro-electric power house, and Inspector on Barker Meadow Dam. *TT 2: P 2.*—June to Nov. 1911 Concrete Foreman, Kerbaugh Constr. Co., on Catskill Aqueduct and Nov. 1911 to Jan. 1912 Constr. Supt., power-transmission lines, Lachine, Que. *TT 0.6: P 0.6.*—Jan. to May 1912 Night Supt., Hydro-Elec. Dam & Power House, Bonny Eagle, Me. *TT 0.3: P 0.3.*—May to Dec. 1912 Gen. Foreman on power-transmission lines, until May for Yadkin River Power Co., Rockingham, N. C., Elec. Bond Share Co., Engrs. and Bldrs., then for Phoenix Constr. Co., Waco, Tex. *TT 0.6: P 0.6.*—Dec. 1912 to June 1914 Gen. Foreman, hydro-electric power plant, Ebro Power & Irrigation Co., Barcelona, Spain, F. S. Pearson, Ltd., Engrs. and Contrs. *TT 1.5: P 1.5.*—1915 Labor Foreman, Jacobs & Davies, Engrs., New York City, on subway construction. *TT 0.5: P 0.5.*—1915 to 1917 Munitions Product Agent of British Munitions Board for Northeastern Dist. of U. S. A. *TT 2: P 2.*—1917 to 1919 with U. S. Army, Aug. 1917 commissioned Capt., Jan. 1918 became Major, and March 1919 Lieut. Col.; acting as Inspection Supervisor, Ordnance Dept. at large (2 months), Asst. Gen. Supervisor, Inspection Sec. (1 month), and Gen. Superv. Inspector (5 1/2 months), Gun Div.; member of Ordnance Officers Examining Board (after Nov. 1917), Inspection Mgr., Boston Dist. Ordnance Office (1 year), member Boston Dist. Claims Board and Ordnance Dist. Chf., Boston Dist. and Chairman, Boston Dist. Claims Board. *TT 2.5: P 2.5: RC 2.5.*—Feb. 1920 to date Constr. Engr., The Detroit Edison Co., in executive charge of Drafting room on plans, and in responsible charge of construction and design of work (about $125 000 000), including construction of addition (75 000 kw.) to Connors Creek plant; Marysville Power House (160 000 kw.), Trenton Channel Power House (300 000 kw.), Delray Power House (150 000 kw.), also substation, warehouse and office building construction. *TT 11.8: P 11.8: RC 11.8: D 11.8.*—*TT 22.2: P 22.2: RC 13.8: D 12.1.* Refers to A. Dow, G. H. Fenkell, J. P. Hallihan, W. S. Housel, L. G. Lenhardt, A. H. Place, F. B. Spencer.

17

(8) FOLSE, JULIUS AUDREY, 1525 East Fifty-third St., Chicago, Ill. (Age 37. Born Labadieville, La.) 1917 B. S. in Eng., and 1920 C. E., Northwestern Univ. *TT 4: P 4.*—Dec. 1917 to Aug. 1919 Airplane Inspector, 826th Aero Squadron, U. S. Army, Romorantin, France, being Private, 1st Class Sergeant, Air Corps, inspecting repairs, riggings, engine installation, etc., on reconstructed war airplanes, particularly DH-4's powered by Liberty motors; Oct.-Dec. 1918 at U. S. Army Engrs. School, Langres, France, became 2d Lieut. *TT 1.7: P 1.7: RC 1: D 1.*—Sept. 1912 to Nov. 1917 (vacations and spare time while student) Computer, June 1920 to Sept. 1924 Research Asst., being Prin. Asst. to Dean of School of Eng., Northwestern Univ., and July 1926 to Sept. 1927 Research Associate, on research in physics, evaporation, runoff, etc., at Carnegie Inst. of Washington, D. C. *TT 5.5: P 5.5: RC 3.2: D 3.2.*—Oct. to Dec. 1924 Engr., Village of Hinsdale, Ill., investigating and reporting on mechanical condition of municipal light and power plant and preparing recommendations for rehabilitation. *TT 0.3: P 0.3: RC 0.3: D 0.3.*—Jan. 1925 to June 1926 in office of Harza Eng. Co., Chicago, on computations relative to hydrology and hydro-electric design, calculating flood probabilities, evaporation, runoff, water-turbine performance, etc. *TT 1.5: P 1.5: RC 0.5: D 0.5.*—Oct. 1927 to Sept. 1928 Research Asst., Bureau of Business Research, Northwestern Univ. School of Commerce, Chicago, research and teaching in economics. Sept. 1928 to date Curator, Div. of Motive Power and Transportation, Museum of Science and Industry, Chicago, preparing permanent exhibits portraying historic evolution, theory and economics on methods of power generation and transmission, land, water and air transporation. *TT 3: P 3: RC 3: D 3.*—*TT 16: P 16: RC 8: D 8.* Refers to F. A. Dale, J. R. Freeman, N. R. Gibson, L. F. Harza, R. E. Horton, A. F. Meyer, E. S. Nethercut.

18

(10) FRENCH, FREDERICK CARPENTER, 540 South Crest Road, Chattanooga, Tenn. (Age 49. Born Chattanooga, Tenn.) 1905 to date with City of Chattanooga, Tenn., as follows: 1905 to 1907 Chainman, Rodman and Instrumentman, on surveys and drafting; 1907 to 1921 Inspector and Draftsman, on sewer and street paving construction, vitrified pipe, brick and concrete sewers, brick sewer in tunnel, field cast concrete pipe, Inspector on sewage-pumping station (1910-1911) and Grand Drive retaining wall (1915); 1921 to 1928 Chf. Draftsman, assisting on design for sewer systems, 12th and 13th Wards (9 sq. miles), in charge of office work; since 1928 Office Engr., designed system of sewers for part of 18th ward, including steel and concrete creek crossing (150 ft. long), system for 14th and 15th wards, grades for street paving projects, in charge of office work. *TT 18: SP 8: P 10: RC 10: D 3.* Refers to C. A. Betts, W. R. King, J. E. Morelock, A. F. Porzelius, W. H. Sears, W. H. Wilson, D. H. Wood.

19

(11) GARDNER, GEORGE LEON, Jr., 5164 Ruthelen St., Los Angeles, Cal. (Age 24. Born Bedford, Ohio.) 1930 B. S. in A. E., Univ. of Southern Cal. *TT 4: P 4.*—At present post-graduate student, Univ. of Southern California.—*TT 4: P 4.* Refers to R. M. Fox, D. M. Wilson.

20

(5) GOLZE, ALFRED RUDOLF, 3039 Macomb St. N. W., Washington, D. C. (Age 26. Born Washington, D. C.) 1930 B. S. in C. E., Towne Sci. School, Univ. of Pa. *TT 4: P 4.*—Jan. 1925 to Oct. 1927 Jun. Draftsman and summer 1929 Structural Draftsman, Dept. of City Transit, Philadelphia, Pa., making drawings and tracings for underground subway structures and (1 year) on grading computations for terminal yard. *TT 1.3: SP 1.3.*—Oct. 1923 to June 1924 and Oct. 1927 to June 1930 student. June 1930 to date with Bureau of Valuation, Interstate Commerce Comm., until May 1931 as Jun. Engr., on computations for railroad valuation reports and since May 1931 Asst. Engr. on depreciation study of rails, ties, etc. *TT 1.5: P 1.5.*—*TT 6.8: SP 1.3: P 5.5.* Refers to H. C. Berry, W. H. Chorlton, C. A. Hoglund, E. L. Ingram, S. W. Lamborn, C. W. Palmer, W. S. Pardoe.

21

(13) GREENE, GEORGE ARTHUR, Box 1103, Sacramento, Cal. (Age 33. Born Great Falls, Mont.) 1931 B. S., Univ. of Cal. *TT 4: P 4.*—June 1922 to Oct. 1926 with Los Angeles Harbor Dept., until Nov. 1923 successively as Chainman and Instrumentman on surveys, hydrographic and topographic work, staking out wharves and piers, taking soundings, etc., then Chf. of Survey Party staking out for wharves and piers, acted as Topographer, and as Engr. on construction of wharves and piers, had charge of sounding party, surveys, etc. *TT 3.7: SP 0.8: P 2.9: RC 2.9.*—Oct. 1926 to Dec. 1928 Chf. of Survey Party, City Engr's Office, Los Angeles, on street center line surveys and profiles,

staking out for curbs, pavement, storm drains, sewers, etc. *TT 2.2: P 2.2: RC 2.2.*—May to Aug. 1930 and June 1931 to date with State of California, first as Instrumentman, Div. of Highways, Dist. 3, acting as Chf. of Party, etc., on surveys, and since June 1931 Asst. Res. Engr., Bridge Dept., acting as Engr. and Inspector on bridge contracts, being either in charge of a project or of a unit of a large project. *TT 0.7: SP 0.1: P 0.6: RC 0.6.*—*TT 10.6: SP 0.9: P 9.7: RC 5.7.* Refers to C. Derleth, Jr., T. E. Ferneau, F. S. Foote, C. G. Hyde, F. W. Panhorst, H. D. Stover.

22

(4) GROSSART, LEWIS PHAON, 816 Chew St., Allentown, Pa. (Age 37. Born Delano, Pa.) Registered Prof. Engr. & Land Surveyor, State of Pennsylvania. 1914 to 1916 student in Civ. Eng., Lehigh Univ. *TT 1: P 1.*—1908 to 1910 Chainman, Rodman and Draftsman on general municipal work. 1910 to 1915 (while student) Draftsman, Bethlehem Steel Co. (1910, 3 months) in charge of Field Party, street grading, storm sewers and laying cast iron water pipe in three towns (1912, 4 months), in charge of Party, City of Allentown, on sewer construction, preparing estimates and making up assessments (1913 and 1914, 10 months), and Transitman, Lehigh Valley R. R., maintenance of way and rebuilding yards near Jersey City (1915, 4 months). 1916 Transitman, Central R. R. of New Jersey, on valuation work and resetting centre line and right-of-way monuments. *TT 0.2: SP 0.2.*—1917 in charge of Field Party, on municipal construction in three towns and on precise azimuth survey for Bethlehem Steel Co. *TT 0.9: P 0.9: RC 0.9.*—1917 to 1927 (except Nov. 1917 to Feb. 1919 with U. S. Army, Ordnance Dept., on construction of Ordnance Base Depot in France) Engr. in charge of field and office work, water-works layout and extension for Northampton County Water Co. and Hellertown Borough, sanitary sewers for real estate developments, railroad work on cement-mill construction and concrete road work and caisson work on 22-story office building. *TT 9.7: SP 0.7: P 9: RC 7: D 2.*—1928 to 1931 on design, construction and supervision of sanitary sewers and separate sludge digestion system at Northampton, Pa., including pumping stations; plans and supervision of electrification, municipal pumping plant at Catasauqua Water-Works. *TT 4: P 4: RC 3.5: D 0.5.*—*TT 15.8: SP 0.9: P 14.9: RC 11.4: D 2.5.* Refers to L. P. Bailey, R. J. Fogg, R. L. Fox, M. O. Fuller, R. E. Neumeyer, G. H. Shaw, W. L. Wilson.

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(15) HAILE, CHARLES RADCLIFFE, 6725 Sherman St., Houston, Tex. (Age 42. Born San Antonio, Tex.) 1912 B. S. C. E., Agri. and Mech. Coll. of Tex. *TT 4: P 4.*—1909 to 1910 Levelman on railroad surveys. *TT 0.6: SP 0.6.*—1912 to 1913 Asst. Res. Engr., National Rys. of Mexico, on construction of Linea de la Costa del Golfo. *TT 1.5 P 1.5: D 1.5*—March to Aug. 1914 Timekeeper on tunnel construction, Estrada do Fierro Central do Brazil. *TT 0.2: SP 0.2.*—Aug. 1914 to Feb. 1916 Asst. Engr., Rio de Janeiro Light & Power Co., on tunnel construction. *TT 1.5: P 1.5: RC 1.5.*—April 1917 to Aug. 1919 1st Lieut. and Capt., 315th Engrs., 90th Div., U. S. Army. *TT 2.3: P 2.3: RC 1.*—Aug. 1919 to March 1921 Asst. County Engr., Kleberg County, on highway construction. *TT 2.7: P 2.7: D 2.7.*—March 1921 to March 1924 Div. Engr., and March 1924 to March 1925 Constr. Engr., State Highway Dept. of Texas. *TT 4: P 4: RC 4: D 4.*—March 1925 to Feb. 1926 member of firm, Jno. F. Buckner & Co., Bridge Contrs., Cleburne, Tex. *TT 0.9: P 0.9: RC 0.9.*—Feb. 1927 to Feb. 1929 Asst. County Engr. on highway construction, and Feb. 1929 to date County Engr., Harris County. *TT 4.8: P 4.8: RC 2.8: D 4.8.*—*TT 22.6: SP 0.9: P 21.7: RC 10.2: D 13.* Refers to G Gilchrist, J. M. Howe, J. M. Nagle, J. E Pirie, H C. Porter, G. G. Wickline.

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(1) HOFF, LOUIS DALE, 1 Spring Rd., Park Hill, Yonkers, N. Y. (Age 36. Born Rochester, N. Y.) 1921 B. S. in C. E., N. Y. Univ. *TT 4: P 4.*—Summer 1916 Chainman, Rodman and Instrumentman, New York Central R. R.—Dec. 1917 to Dec. 1919 Watch Officer, Instructor in Nautical Astronomy, Navigator (troop ship), U. S. Navy Dept., in responsible charge of engineering work, compass correction, soundings, etc. *TT 0.7: P 0.7: RC 0.7.*—March 1922 to July 1924 Asst. Engr., Adirondack Power & Light Corporation, plotting surveys, being Chf. of survey party on concrete design and construction survey, in responsible charge of layout and design of power plant alteration and addition. *TT 2.4: P 2.4: RC 1: D 1*—Oct. 1924 to Nov. 1925 Asst. Engr., Westchester County Park Comm., mapping and designing septic sewerage system and concrete pavement for Saw Mill River Parkway. *TT 1: P 1: RC 0.2.*—Nov. 1925 to Aug. 1931 ill; did clerical and other work, including about 6 months of engineering. *TT 8.1: P 8.1: RC 2: D 1.* Refers to H. E. Breed, C. A. Garfield, E. G. Hooper, C. T. Schwarze, C. H. Snow, D. S. Trowbridge, C. Voetsch.

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(8) **HONOUR, WILFRED MAIN,** Univ. of Illinois, Urbana, Ill. (Age 23. Born Atlanta, Ga.) 1929 B. S. in C. E., Ga. School Tech. *TT 4: P 4.*—July 1929 to Jan. 1930 Designer with Robert S. Fiske, Cons. Engr., Atlanta, Ga., on steel and concrete building design. *TT 0.5: P 0.5.*—Jan. 1930 to date Special Research Graduate Asst. in Civ. Eng., Univ. of Illinois, research on multiple-span concrete arches. *TT 1.7: P 1.7.*—*TT 6.2: P 6.2.* Refers to H. Cross, R. S. Fiske, W. C. Huntington, F. C. Snow, W. M. Wilson.

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(7) **HUGHES, MERVIN MARK,** Sault Ste. Marie, Mich. (Age 27. Born East Jordan, Mich.) Sept. 1929 to June 1931 student in Civ. Eng., Univ. of Illinois. *TT 1: P 1.*—June 1926 to March 1927 contracted gravel and concrete hauling on roads in Illinois, Missouri and Texas for K. Leo Mingus, Inc., Des Moines, Iowa, and Smith Bros. Co., Dallas, Tex.—Dec. 1927 to April 1928 in charge of rigging and equipment for Sumner Sollitt Co., on construction of U. S. War Veterans Hospital, Tucson, Ariz. *TT 0.3: P 0.3.*—June 1924 to date (except as above noted) with Price Bros. Co., until July 1925 as Labor Foreman in charge on construction of a hydro-electric plant; July to Dec. 1925 Master Mechanic, and Dec. 1925 to June 1926 Labor Foreman, on construction of hydro-electric plant for Eastern Iowa Power Co., assisted in setting James Leffel water wheels, had charge of repair and maintenance, etc.; March to Dec. 1927 installed automatic operating equipment, and built emergency spillway on Dam No. 1, near New Braunfels, Tex., for Fargo Eng. Co., Jackson Mich.; July 1928 to Feb. 1929 Constr. Foreman on steam power plant at Evansville, Ind., for Commonwealth Edison Co.; June to Oct. 1930 in charge of construction of Red Wood Pipe Line on hydro-electric development at Victoria, Mich., for Copper Range Power Co.; since Nov. 1931 on hydro-electric development at Sault Ste. Marie, Mich., for Sault Edison Co. *TT 3.3: SP 0.4: P 2.9.*—*TT 4.6: SP 0.4: P 4.2.* Refers to E. L. Chandler, C. P. Hughes, A. Jorgensen, M. J. Orbeck, W. H. Rayner, C. C. Wiley.

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(15) **JOHNSON, CLARENCE WEST,** 916 Sixth St., Port Arthur, Tex. (Age 37. Born Pittsboro, Miss.) June 1912 to Jan. 1913 with G. L. Green, C.E., Aberdeen, Miss., as Instrumentman on level, transit and some office work. *TT 0.2: SP 0.2.*—Jan. 1913 to Jan. 1914 Instrumentman and Inspector, County Board of Drainage Commrs., Tippah County, Miss. *TT 0.5: SP 0.5.*—Jan. 1914 to March 1915 with J. E. Brown, C.E., Kosciusko, Miss., until July 1914 as Instrumentman on highway location and construction, then Supt. of team outfit and right-of-way clearing on highway construction. *TT 0.6: SP 0.6.*—March to Sept. 1915 Levelman on railroad survey, Yazco City to Carthage, Miss. *TT 0.2: SP 0.2.*—Sept. 1915 to Aug. 1916 County Surveyor, Calhoun County, Miss. *TT 0.9: SP 0.2: P 0.7.*—Aug. to Oct. 1916 Transitman, South Western Constr. Co., Memphis, Tenn., on railroad survey. *TT 0.1: SP 0.1.*—Oct. 1916 to Jan. 1917 Drainage Engr., County Board of Drainage Commrs., Yalobusha County, Miss., in charge of survey and design of drainage system for Otucalofa Drainage Dist. *TT 0.3: P 0.3: RC 0.3.*—Jan. to July 1917 Chf. of Party for Alf. A. Oldfield, Drainage Engr., Huntington, Tenn., on drainage survey. *TT 0.5: P 0.5: RC 0.5.*—July 1917 to Jan. 1926 with Associated Eng. Co., Cons. Engrs., Memphis, Tenn., in charge of surveys and design of drainage systems. *TT 8.5: P 8.5: RC 4: D 4.5.*—Jan. to June 1926 Locating Engr., Jackson & Eastern R.R. Co. *TT 0.5: P 0.5: D 0.5.*—June 1926 to April 1927 Res. Engr., Gulf, Mobile & Northern Ry., on construction. *TT 0.8: P 0.8: RC 0.8.*—April 1927 to Feb. 1928 Drainage Engr., Leake County Drainage Dist., on survey and design of drainage system. *TT 0.8: P 0.8: RC 0.8: D 0.1.*—Feb. to Nov. 1928 Highway Engr., Lena Road Dist., on survey, plans and construction. *TT 0.8: P 0.8: RC 0.8: D 0.1.*—Nov. 1928 to March 1929 Res. Engr., Morgan & Co., Cons. Engrs., Jackson, Miss., on water-works and sewer construction. *TT 0.3: P 0.3: RC 0.3.*—March 1929 to date Chf. Engr., Coastal Constr. Co., Orange, Tex., in charge of construction; has built sewer, water-works, gas and oil pipe lines, canals, docks and bridges; had charge of construction of welded battle deck steel bridge at Port Arthur, Tex. *TT 2.6: P 2.6: RC 2.3: D 0.3.*—*TT 17.6: SP 1.8: P 15.8: RC 9.8: D 5.5.* Refers to N. E. Colburn, A. S. Fry, J. A. Harman, C. W. Hughes, L. W. Mashburn, C. W. Okey.

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(12) **KAHL, FRANK BUDD,** 305 East Fifty-second North, Portland, Ore. (Age 24. Born Trenton, N. Dak.) 1929 B. S. in C. E., Ore. State Coll. *TT 4: P 4.*—Summer 1927 Rodman, Great Northern Ry.—Summer 1928 and June 1929 to date with Spokane, Portland & Seattle Ry., first as Field Computer, estimating quantities for road valuation; June to Sept. 1929 Draftsman; Sept. 1929 to Nov. 1930 Inspector in field charge of five large frame trestle renewals (under traffic), giving lines, grades and estimates; Nov. 1930 to

July 1931 in office of Bridge Engr., designing, detailing and drafting timber and steel bridges and making bills of material and estimates (steel design consisted of plate girders, Warren trusses, viaduct towers and a 250-ft. drawbridge); since July 1931 in field charge of renewing two pile trestles (30 000 lin. ft., under traffic) and (since Nov. 1931) of erection of 328-ft. steel bridge on concrete piles (under traffic), consisting of 115-ft. Warren truss, plate girders and viaduct towers. *TT 2.4: SP 0.1: P 2.3: RC 1.5: D 0.8.*—*TT 6.4: SP 0.1: P 6.3: RC 1.5: D 0.8.* Refers to W. E. Burkhalter, B. M. Howard, L. K. Needham; H. S. Rogers, C. F. Thomas.

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(3) **KOHL, LEWIS EUGENE,** 120 Reynolds Arcade, Rochester, N. Y. (Age 40. Born Rochester, N. Y.) Licensed Prof. Engr. and Land Surveyor, New York State.—Student, Mechanics Inst., Rochester, N. Y., and in Civ. Eng. course, International Correspondence Schools.—1908 to 1925 (except Sept. 1917 to Aug. 1918) with City Engr.'s Office, Rochester, N. Y., 4 years as Chairman and Rodman on construction of Cobb's Hill Reservoir, Central Ave. Bridge, sewers, pavements, etc.; 12 years Rodman, Transitman and Chf. of Party on surveys and monuments; 1 year Chf. of Party on surveys at Hemlock, Canadice and Conesus Lakes, establishing property corners, taking soundings, etc. *TT 13: SP 4: P 9: RC 9.*—Sept. 1917 to Aug. 1919 Engr. in charge of construction for Crowell, Lundorf & Little, Contrs. on U. S. Govt. amunition plant (forge and small arms plants), Rochester, N. Y. *TT 1: P 1: RC 1.*—1925 to 1927 member of firm, Weeks & Kohl, Civ. Engrs., supervised design and construction of East Gates Water Dist., and Summerville Dist. sewer and water additions, and on engineering for Town of Gates, being in charge of design, improvements, supervision, etc. *TT 2: P 2: RC 2: D 0.5.*—1928 to date Town Engr. of Gates, in charge of all engineering; designed sewers, pavements and sidewalks for Rellim Heights, Renouf Heights, Rosecroft and Frostholm Subdivisions, etc.; made surveys and prepared plans for proposed railroad grade crossings in Town of Gates (Howard, Statt and Coldwater Roads); also in private practice of engineering and surveying at Rochester, on work for other towns, villages, etc. made surveys and maps of Gates for Planning and Zoning Ordinance. *TT 3.9: P 3: RC 2: D 1.*—*TT 19: SP 4: P 15: RC 14:* D 1.5. Refers to E. A. Fisher, G. P. Hevenor, C. C. Hopkins, W. S. Lozier, L. B. Smith, A. L. Vedder, G. C. Wright.

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(4) **LIEBSCH, JOSEPH PETER HARRY,** 6815 North Gratz St., Philadelphia, Pa. (Age 42. Born Philadelphia, Pa.) 1911 B. S. in C. E., Towne Sci. School, Univ. of Pa. *TT 4: P 4.*—June to Aug. 1911 Structural Detailer, American Bridge Co., Edge Moor, Del. *TT 0.2: P 0.2.*—Aug. to Dec. 1911 Rodman, High-Pressure Fire Service System, Dept. of Public Safety, Philadelphia, Pa., on laying of high-pressure mains. *TT 0.2: SP 0.2.*—Dec. 1911 to May 1912 Draftsman, Standard Meter Co., on design and details of gas and station meters. *TT 0.4: P 0.4.*—May 1912 to Nov. 1913 with Bureau of Surveys, Dept. of Public Works, Philadelphia, until July 1913 as Draftsman on city planning and grade-crossing elimination, then Transitman on municipal surveys. *TT 1.2: SP 0.1: P 1.1.*—Nov. 1913 to Aug. 1920 with Dept. of City Transit, Philadelphia, Pa., until Jan. 1918 as Structural Draftsman on details and designs of subway and elevated railway structures, then Asst. Engr., underpinning and related construction on Frankford Elevated Ry. *TT 6.8: P 6.8: D 4.2.*—Aug. 1920 to date Structural Designer and Checker, Bridge Div., Bureau of Eng. and Surveys, on design of bridges, masonry structures and foundations; also taught engineering (nights) in Central and William Penn High Schools (Oct. 1919 to date), Temple University (Sept. 1931 to date) and Drexel Inst. (Jan. to Feb. 1925); past 10 years also in private practice, designed and had responsible charge of several bridge projects for cities and boroughs near Philadelphia. *TT 11.3: P 11.3: RC 7: D 11.3.*—*TT 24.1: SP 0.3: P 23.8: RC 7: D 15.5.* Refers to J. E. Boatrite, S. H. Noyes, H. H. Quimby, E. R. Schofield, F. G. Schworm, S. M. Swaab.

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(4) **MODJESKI, CHARLES EMANUEL JOHN,** 1420 Walnut St., Philadelphia, Pa. (Age 35. Born Chicago, Ill.) 1921 C. E., Cornell Univ. *TT 4: P 4.*—April to July 1915, April 1921 to July 1922 and Sept. 1923 to Jan. 1924 with Ralph Modjeski, successively as Mill and Shop Inspector in Steelton Plant of Pennsylvania Steel Co., on steel for Memphis Bridge, Inspector on reconstruction of Cincinnati, New Orleans & Texas Pacific Bridge across Ohio River at Cincinnati, Ohio, and Res. Engr. in charge of borings into foundations of old railroad bridge across Hudson River at Poughkeepsie, N. Y. *TT 1.4: SP 0.5: P 0.9: RC 0.4.*—Jan. 1924 to Sept. 1927 with Frank M. Masters, Harrisburg, Pa., as Chf. Draftsman in charge of design and Res. Engr. in charge of construction of concrete arch bridge at Clark's Ferry, Pa. (about 9 months), and of Market St. Bridge at Harrisburg, Pa. (over 2 years), both across Susquehanna River and after Oct. 1926

Chf. Draftsman in charge of designs and estimates for proposed bridge across Ohio River at Louisville, Ky., recomputation of stresses, Niagara-Clinton Arch, Niagara Falls, N. Y., and reconstruction of State St. Bridge over railroad tracks at Harrisburg. *TT 3.7: P 3.7: RC 3.7: D 2.9.*—Feb. 1928 to Dec. 1929 Res. Engr. with Modjeski, Masters & Chase, on Tacony-Palmyra Bridge across Delaware River, Philadelphia. *TT 1.9: P 1.9: RC 1.9.*—Oct. 1927 to Jan. 1928 and Jan. 1930 to date with Modjeski & Chase, first on design of main arch ribs, Henry Ave. bridge across Wissahickon Valley, Philadelphia, and since Jan. 1930 Res. Engr. on Henry Ave. bridges over Philadelphia & Reading tracks at 30th St. across Wissahickon Valley, Philadelphia. *TT 2.3: P 2.3: RC 2.3: D 0.3.*—*TT 13.3: SP 0.5: P 12.8: RC 8.3: D 3.2.* Refers to M. B. Case, C. E. Chase, F. M. Masters, R. Modjeski, H. J. Sherman, G. B. Woodruff.

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(9) **MORRIS, WILLARD FRANK,** 1047 Sunbury Rd., Columbus, Ohio. (Age 24. Born Columbus, Ohio.) 1929 B. S. in C. E., and 1930 C. E., Princeton Univ. *TT 4: P 4.*—June 1930 to June 1931 with Goodyear Zeppelin Corporation, 6 months as Draftsman in Structural Design Dept. and 6 months Designing Engr. in Stress Analysis Dept. *TT 1: P 1.*—*TT 5: P 5.* Refers to G. E. Beggs, F. H. Constant.

33

(1) **O'DONNELL, JAMES PATRICK,** 117-34 One Hundred Fortieth St., South Ozone Park, N. Y. (Age 23. Born New York City.) 1930 B. S. in C. E., Cooper Union Inst. Tech. 1931 C. E., Univ. of Ill. *TT 4: P 4.* Refers to F. E. Foss, W. C. Huntington, G. Morrison, T. C. Shedd, J. P. J. Williams.

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(13) **PAGET, FREDERICK HILTON,** 504 Twenty-first St., Sacramento, Cal. (Age 33. Born Hazel Hill, N. S., Canada.) 1920 B. S., Univ. of Cal. *TT 4: P 4.*—July 1918 to Aug. 1919 First Lieut., 11th Battalion, Canadian Engrs., overseas. *TT 0.5: SP 0.5.*—Dec. 1920 to March 1921 Computer, Sutter Basin Co., Knights Landing, Cal., on subdivision work. *TT 0.1: SP 0.1.*— March 1921 to May 1922 Draftsman, Computer and Instrumentman, Reclamation Dist. No. 2047, Colusa, Cal. *TT 0.6: SP 0.6.*—May 1922 to April 1923 and April 1924 to May 1927 with Fred H. Tibbetts, San Francisco, until April 1923 as Draftsman, Computer and Instrumentman on irrigation, drainage and subdivision work, and after April 1924 Draftsman, Computer, Designer, Chf. of Party and Asst. Engr. on development of reclamation and irrigation districts, including reconnaissance, surveys, design of irrigation and drainage systems, with incidental small structures, and compilation of estimates and reports. *TT 3: SP 0.9: P 2.1: RC 1.5: D 0.6.*—April to July 1923 Computer with Peter R. Gadd, Sacramento, Cal., on triangulation, made computation for triangulation survey of 21 000-acre ranch. *TT 0.2: P 0.2.*—July to Sept. 1923 with W. E. Callaghan Constr. Co., Marysville, Cal., in charge of clearing brush from flood-control levee right-of-way and erecting temporary power-line poles. *TT 0.1: SP 0.1.*—Sept. to Nov. 1923 Instrumentman, P. B. Eng. Constr. Co., Los Angeles, set slope stakes to keep drag lines to line the grade on flood-control levees. *TT 0.2: P 0.2.*—Nov. 1923 to April 1924 and May 1927 to June 1928 with County Engr., Monterey County, Cal., first as Transitman on road location and subdivision work, and after May 1927 Chf. of Party on road location, estimates, subdivision, small farm irrigation layouts, etc. *TT 1.5: P 1.5: RC 1.5.*—Aug. 1928 to July 1929 Chf. Draftsman, Tax Factors, Inc., on appraisal of San Diego County, in charge of Rural Drafting Dept. (under Office Engr.). *TT 1: P 1.*—July to Dec. 1929 private work, appraisals of small homes and suburban lots for building and loan companies and banks in San Diego. *TT 0.4: P 0.4: RC 0.4.*—Jan. 1930 to date with State of California, until Feb. 1931 as Jun. Hydr. Engr. on field reconnaissance and office studies for salinity control in Sacramento River Delta and the projected salt water barrier, then Jun. Engr. of hydraulic structure design, involving investigating capacities of dam spillway and stresses developed in various types of dams under operating conditions; since Aug. 1931 Asst. Engr. of hydraulic structure design, involving measuring water directed from Sacramento River between Redding and Knights Landing, Cal. *TT 1.9: P 1.9.*—*TT 13.5: SP 2.2: P 11.3: RC 3.4: D 0.6.* Refers to E. E. Blackie, G. E. Goodall, G. W Hawley, R. Matthew, S. H. Searancke, H. M. Stafford, F. H. Tibbetts, R. G. Wadsworth.

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(2) **PALMER, WAYNE FRANCIS,** 129 Morgan St., Holyoke, Mass. (Age 36. Born Ft. Wayne, Ind.) 1917 B. S., Dartmouth Coll. *TT 2: P 2.*—March to Sept. 1917 student, U. S. Naval Academy.—Sept. 1917 to May 1918 Deck Officer, in charge of all communications, U. S. S. Wyoming, being Ensign, Lt. j. g. and Sr. Lieut., designed and built models

of new type of submarine mine and counterminer. *TT 1.5: P 1.5: D 1.5.*—May 1918-Dec. 1921 with Bureau of Ordnance, in charge of Mine Laboratory, creating, designing and building submarine mines, depth charges, launching mechanisms, pyrotechnics, fuzes, superintended and designed equipment for tests on action of mines in currents and tideways. *TT 3.5: P 3.5: RC 3.5: D 3.5.*—March 1922 to date with The Palmer Steel Co., steel fabricators, Springfield, Mass., until April 1928 as Salesman and in charge of contracting, and since then Gen. Mgr., in charge of all departments. *TT 6.7: SP 3: P 3.7: RC 3.7.*—*TT 13.7: SP 3: P 10.7: RC 7.2: D 5.* Refers to M. F. Brown, H. Kornfeld, E. E. Lochridge, G. P. O'Connell, G. E. Pellissier, R. A. Smith, M. M. Thrane, A. C. Waghorne.

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(10) PRICHARD, MASON CARTER, 3522 Randall St., Jacksonville, Fla. (Age 32. Born Mobile, Ala.) 1923 B. S., Univ. of Ala. *TT 4: P 4.*—June 1918 to June 1919 Transitman, Chickasaw Shipbuilding Co., on building layout, railroad location and topographical surveys. *TT 0.5: SP 0.5.*—June to Dec. 1919 Field Engr., C. S. Co. (U. S. Steel Corporation), on power-plant construction, lines, grades, estimates, inspection of materials and workmanship, etc. *TT 0.2: SP 0.2.*—June 1924 to May 1925 with Alabama Highway Dept., until Oct. 1924 as Draftsman on design and plans, then Asst. Engr. on Federal Aid work, in charge of bridge and culvert construction on 26-mile project. *TT 1: P 1: RC 0.7: D 0.3.*—May 1925 to May 1927 with Butler, Barnett & Taylor, Cons. Engrs., West Palm Beach, Fla., until April 1926 as Engr. in field and office on municipal subdivision and general engineering, then Office Engr. in charge of design, drafting, estimates, specifications, plans for concrete bridges, street paving, sea walls, etc. *TT 2: P 2: RC 2: D 1.1.*—March to Nov. 1928 in private practice at Mobile, Ala., plans for and supervision of construction of mill buildings, bulkheads, surveys, appraisals, etc. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—Dec. 1919 to Sept. 1920, May 1927 to Feb. 1928 and Nov. 1928 to date with U. S. Engr. Office, first as Design Draftsman at Tuscaloosa, Ala., on concrete design, design of locks and dams, mechanical drafting and assisting in compiling and assembling report on engineering studies; May 1927 to Feb. 1928 Asst. Engr. at Chattanooga, Tenn., special investigations of Cove Creek hydro-electric and reservoir project, assisted on designs and plans for power plant, spillways and locks; made stability analysis of gravity dam section; assisted in design and plans for number of projects on Clinch and other rivers; Nov. 1928 to Nov. 1930 Asst. and Associate Engr. at Mobile, Ala., in responsible charge of surveys, mapping and drafting report on surveys of Warrior and Tombigbee Rivers, for plan for combined developments of hydro-electric power, navigation and flood protection; directed design of locks, dams, spillways, etc.; since Nov. 1930 at Jacksonville, Fla., as Engr. in charge of preliminary examination and survey of trans-Florida ship canal under special board of Army Engrs., having complete supervision and direction of hydraulic, geological, commercial, and physical investigations and surveys and drafting of report. *TT 4.3: SP 0.4: P 3.8: RC 3.8: D 4.1.*—*TT 12.7: SP 1.1: P 11.6: RC 7.3: D 6.2.* Refers to J M. Boyd, W. H. Goodloe, G. B. Hills, S. C. Houser, R. D. Jordan, C. E. McCashin, G. F. Whittemore, G. A. Youngberg.

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(1) QUIRICONI, EUGENE, 27 Bedford St., New York City. (Age 23. Born Camaiore, Italy.) 1929 B. S., and 1930 C. E., Coll. of City of N. Y. *TT 4: P 4.*—Summer 1927 Eng. Asst., Gibbs & Rice Constr. Corporation, Bldg. Contrs., giving line and grade on building foundations.—Oct. 1927 to Jan. 1929 Student Instructor in Surveying, Coll. of City of New York.—March to April 1920 Steel Estimator, Maxwell Spiro & Co., Jersey City, N. J. *TT 0.1: P 0.1.*—April to May 1930 and Oct. 1930 to Jan. 1931 Instructor in surveying, mathematics and drafting, Mondell Eng. Inst., New York City. *TT 0.4: P 0.4.*—May to Oct. 1930 Material Clerk with Jackson & Moreland, Hoboken, N. J., on Delaware, Lackawanna & Western R.R. work, made takeoffs from prints for purchasing materials. *TT 0.4: P 0.4.*—Jan. to May 1931 Asst. Supt., Raymore Constr. Co., on school building construction. *TT 0.3: P 0.3.*—May 1931 to date Structural Steel Draftsman, Grade 3, Board of Transportation, New York City, drafting and design. *TT 0.7: P 0.7.*—*TT 5.9: P 5.9.* Refers to R. E. Goodwin, F. O. X. McLoughlin.

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(4) RANLETT, CHARLES AUGUSTUS, Jr., Sharon Apts., Sharon Hill, Pa. (Age 27. Born Cambridge, Mass.) Feb.–May 1926 Chainman, Dwight P. Robinson, Inc., Boston, Mass., on surveys. *TT 0.2: SP 0.2.*—June–Oct. 1926 Rodman and Instrumentman, Gibbs & Hill, Inc., New York City, on surveys, cross sections, staking and setting for Catenary supports for railroad electrification (about 20 miles). *TT 0.2: SP 0.2.*—Nov. 1926 to Feb. 1928 Rodman, Transitman and Leveler with Day & Zimmerman, Philadelphia, Pa., on surveys,

location, cross sections, staking of towers, lines and grades for earth and concrete footings, checking road and ground clearance, transmission line, Conowingo, Md., to Philadelphia. *TT 0.6: SP 0.6.*—Feb. to April 1928 Chf. of Party with John T. Brown, Surveyor, Billerica, Mass., on surveys and subdivisions, plotting and tracing same. *TT 0.2: P 0.2: RC 0.2.*—April 1928 to Feb. 1930 with Boston & Maine R. R., until Feb. 1929 as Chainman and Rodman, Portland Div., Dover, N. H., being Concrete Inspector on bridge construction, Instrumentman giving line and grade, cross sections, estimating quantities, bridge construction, line and grade for new track and rock ballast, track and bridge surveys, and last year Rodman and Transitman, Constr. Dept., Boston, Mass., being Instrumentman, on line and grade inspecting, figuring quantities for bridge construction, Party Chf. on surveys and construction in yard, line, grade, quantities and reports on highway and railroad bridge construction. *TT 1.1: SP 0.7: P 0.4: RC 0.4.*—Feb. to April 1930 Materialman, United Engrs. and Constrs., Inc., Philadelphia, Pa., checking and distributing material from field offices at Norristown and South Langhorne, Pa. *TT 0.1: SP 0.1.*—April 1930 to date Chf. of Party, with Damon & Foster, Engrs., Sharon Hill, Pa., on survey for pipe lines, Marcus Hook, Pa., to Ohio State line and Marcus Hook to Syracuse, N. Y., property surveys and inspection of road and sewer construction, (1930) lines and grades and Chf. Inspector on 11 miles sewerage system (1931). *TT 1.7: P 1.7: RC 1.7.*—*TT 4.1: SP 1.8: P 2.3: RC 2.3.* Refers to E. W. Backes, H. E. Boardman, N. Foster, J. T. Kiernan, D. B. Zentmyer.

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(16) RENO, SAMUEL HAROLD, Chanute, Kans. (Age 29. Born Scammon, Kans.) 1923 to 1926 Special student, Eng. School, Kansas State Teachers Coll.—1918 to 1921 Chainman and Rodman, Missouri, Kansas & Texas Ry., Office of Engr., M. of W., maintenance, bank widening, curve and alignment. *TT 1.2: SP 1.2.*—1921 to 1923 Instrumentman, Draftsman and Asst. County Engr., Cherokee County. *TT 1.3: SP 1: P 0.3: RC 0.3.*—1924 to 1925 Office Engr., Missouri State Highway Dept., on road plans. *TT 0.5: P 0.5: D 0.5.*—1926 to date with Kansas Highway Comm., until 1929 as Res. Engr., Crawford and Cherokee Counties, on earthwork, culverts, bridges and paving, and since 1929 Maintenance Supt.. *TT 5.2: P 5.2: RC 5.2.*—*TT 8.2: SP 2.2: P 6: RC 5.5: D 0.5.* Refers to H. D. Barnes, W. V. Buck, F. W. Epps, C. I. Felps, J. F. Grady, A. A. Laird, I. E. Taylor, R M. Willis.

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(13) ROGERS, FRANKLIN, 1430 Larkin St., San Francisco, Cal. (Age 43. Born Evanston, Ill.) Structural Engr., State of Illinois. Student, Armour Inst. (1909 to 1911) and Chicago Tech. (2 years, evenings). *TT 1: P 1.*—June 1912 to July 1913 Rodman and Transitman with Horn & Smith, Civ. Engrs., Boise, Idaho. *TT 0.5: SP 0.5.*—Sept. 1913 to June 1914 Instructor in mechanical drawing and manual training, Raymond (Wash.) Public School.—July 1914 to Feb. 1916 Draftsman for various architects in San Francisco. *TT 0.5: SP 0.5.*—Feb. 1916 to Jan. 1919 with Corrugated Bar Co., Chicago, Ill., detailing and designing reinforced concrete buildings, estimating reinforcing steel, 1 year in charge. *TT 2: SP 1: P 1: RC 1: D 1.*—Jan. to July 1919 Structural Engr. with A. S. Alschuler, Archt., Chicago, designing steel and reinforced concrete buildings. *TT 0.5: P 0.5: RC 0.5: D 0.5.*—July 1919 to July 1921 and July 1923 to July 1925 with Lockwood Greene & Co., Chicago, until July 1921 as Structural Engr., designing reinforced concrete factory and mill buildings, some structural steel design and after July 1923 Structural and Superv. Engr., designing, specifications, letting contracts and supervising construction. *TT 4: P 4: RC 4: D 4.*—July 1921 to July 1923 Chf. Engr., Kalman Steel Co., Chicago, in charge of all engineering work in drafting room, designing and detailing reinforced concrete buildings. *TT 2: P 2: RC 2: D 2.*—July 1925 to July 1926 Structural Engr. with Wm. H. Warner, designing reinforced concrete hotels, apartment buildings, churches, garages, etc. *TT 1: P 1: RC 1: D 1.*—July 1926 to April 1930 in private practice, designing, detailing and supervising construction of all types of reinforced concrete structures, designed over 100 buildings, up to 20 stories in height. *TT 3.7: P 3.7: RC 3.7: D 3.7.*—April 1930 to Oct. 1931 Structural Engr. with Allen & Garcia, commission in Tomsk, U. S. S. R. (Siberia), designing and detailing coal tipples, bunkers, etc. *TT 1.5: P 1.5: RC 1.5: D 1.5.*—*TT 16.7: SP 2: P 14.7: RC 13.7: D 13.7.* Refers to G. C. Breckinridge, A. P. Clark, A. E. Cummings, S. B. Lincoln, A. E. Lindau, F. M. White.

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(10) SALMON, FRED ARCHER, Sr., P. O. Box 93, Guin, Ala. (Age 31. Born Dadeville, Ala.) March to June 1918 Tracer, and Feb. to Sept. 1919 Rodman Tennessee Coal, Iron & R. R. Co., Birmingham, Ala. *TT 0.4: SP 0.4.*—June 1918 to Feb. 1919 and Sept. 1919 to April 1920 Rodman and Draftsman, Southern Ry. *TT 0.6: SP 06.*—April 1920 to

date with Alabama State Highway Dept., until May 1921 as Rodman and Draftsman; May 1921 to April 1924 Draftsman and Designer on plans; April 1924 to March 1925 Transitman on location and (part of time) Acting Chf. of Party; March 1925 to Sept. 1927 Chf. of Party on location; since Sept. 1927 Res. Engr. in charge of construction (62 2/5 miles) and paving (23 2/5 miles) of road in conjunction with location and plan design for approx. 95 miles. *TT 10: SP 1.7: P 8.3: RC 6.8: D 5.3.*—*TT 11: SP 2.7: P 8.3: RC 6.8: D 5.3.* Refers to H. D. Burnum, W. Finnell, H. H. Houk, R. D. Jordan, J. W. Martin, J. H. Mayer, H. C. Wells.

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(9) **SCHUMANN, GEORGE EDGARD,** 1200 Bender Ave., Cleveland, Ohio. (Age 32. Born in Luxemburg.) 1924 C. E., Inst. of Tech., Zurich. *TT 4: P 4.*—March to Oct. 1924 with Evence Coppee, Brussels, Belgium, field work for construction of 60 coke ovens, being Asst. to Supt. and acting as Interpreter. *TT 0.6: P 0.6.*—Oct. 1924 to June 1926 Designer and Estimator, Anciens Etablissments Paul Wurth, Luxemburg, responsible for design of five bridges, coal tipples, sheet-mill plant in Dudelange and cranes. *TT 1.7: P 1.7: RC 1.5: D 1.7.*—July 1926 to April 1929 Structural Steel Designer, Dravo Contr. Co., Neville Island, Pittsburgh, Pa., designing structural work, derrick boats, cranes, barges, ore bridge, etc. *TT 2.8: P 2.8: D 2.8.*—April 1929 to date Designer, Arthur G. McKee Co., Cleveland, Ohio, on structural steel and reinforced concrete, including design of foundations, gas-producer building, bins and conveyor gallery, sewers, etc. for Newton Steel Plant, Monroe, Mich., of platform and top structure for Davidson Co., Neville Island, of blast furnace columns, stove foundations, blast furnace platform and top structure, skip bridge, dust-catcher and gas-washer foundations, gas-main supports, limestone-crushing building (steel and timber) and conveyor galleries, on Russian plant for magnitogorst, checking design of conveyor bridges and skip bridge for Hamilton Works, Steel Co. of Canada, etc. *TT 2.7: P 2.7: D 2.7.*—*TT 11.8: P 11.8: RC 1.5: D 7.2.* Refers to R. L. Barbehenn, T. K. Clarke, F. Feher, P. J. Jorgensen, B. R. Magee, R. Sailer.

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(6) **SEDER, WILLARD JAMES,** 623 Greendale Ave., Edgewood, Pa. (Age 31. Born Menomonie, Wis.) 1921 B. S. in C. E., Univ. of Wis. *TT 4: P 4.*—June to Aug. 1921 Instructor, Civ. Eng. Dept., Extension Div., Univ. of Wisconsin. *TT 0.1: P 0.1.*—Jan. to June 1922 Computing Clerk, U. S. Forest Products Laboratory, Madison, Wis., computing data on study of waste at saw mills. *TT 0.2: SP 0.2.*—July 1922 to date with McClintic-Marshall Co., until Dec. 1924 taking student training course, detailing structural steel for office and mill buildings, etc. (4 months), Timekeeper on erection of structural steel for steel sheet mill buildings, locomotive shops, Alfred H. Smith Memorial Bridge over Hudson River, Washington Crossing Bridge, Pittsburgh, Pa. (designed erection equipment for 3 hinged arch spans) (1 5/6 years), and Structural Engr. designing steel structures (4 months); Dec. 1924 to Aug. 1925 Structural Engr., preparing estimates of contemplated projects and designing mill buildings, viaducts, truss spans, etc.; since Sept. 1925 at Rankin Works, until March 1931 as Asst. Mgr., being responsible to Works Mgr. for operations, and since March 1931 Mgr. in full charge of operations, involving detailing and fabricating truss and girder bridges, mill and office buildings, etc., including Outerbridge Crossing for Port of New York Authority, Ambassador Bridge from Detroit, Mich., to Windsor, Canada, mill buildings for Great Lakes Steel Corporation, Detroit, and part of Empire State Bldg., New York City. *TT 8.6: SP 0.8: P 7.8: RC 6.6: D 2.5.*—*TT 12.9: SP 1: P 11.9: RC 6.6: D 2.5.* Refers to G. A. Caffall, P. A. Franklin, G. L. Gaiser, J. Jones, G. L. Taylor.

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(1) **SHELDON, ROBERT CLINTON,** Box 1082, Balboa, Canal Zone. (Age 27. Born Findlay, Ohio.) 1926 B. S. in C. E., Ohio Northern Univ. *TT 4: P 4.*—May to Nov. 1925, May 1926 to May 1927 and Sept. 1927 to Nov. 1928 Deputy County Surveyor, Monroe County, Ohio, acting as Asst. Engr. on county highways. *TT 2: SP 0.2: P 1.8: RC 1.1 D 0.5.*—May to Sept. 1927 Draftsman and Instrumentman, State Highway Dept. of Wyoming. *TT 0.2: SP 0.2.*—Nov. 1928 to date with Section of Surveys, The Panama Canal, until March 1930 as Surveyor, and since then Jun. Engr., in charge of party on surveys and topographical mapping in connection with operation and maintenance of The Panama Canal and Panama R. R. *TT 2.7: SP 0.3: P 2.4: RC 1.5.*—*TT 8.9: SP 0.7: P 8.2: RC 2.6: D 0.5.* Refers to G. H. Elbin, W. B. Godfrey, E. S. Randolph, J. L. Schley, A. R. Webb.

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(4) **SHUTE, JAMES SELDEN,** 125 West Olney Ave., Philadelphia, Pa. (Age 51. Born Clarksboro, N. J.) Graduated in Mathematics, Drexel Inst. (1906) and in Eng., Brooklyn

(N. Y.) Evening Technical School (1913).—March 1899 to March 1903 Student and Asst. with Wm. M. Carter, C. E., Woodbury, N. J., on surveys, computation and plots of farm, city properties, trolley lines, improved highways and sewer system. *TT 2.9: SP 1: P 1.9.* —March to April 1903 Transitman for I. W. Hubbard, Philadelphia, Pa., on railroad survey. *TT 0.1: P 0.1.*—April 1903 to May 1909 Transitman and Asst. Engr., Philadelphia Rapid Transit Co., on construction of Market St. Subways, first on surveys, later measuring and computing, and finally in charge of relocation of underground structures and preparing record plans. *TT 6.2: P 6.2: RC 3.*—May 1909 to April 1910 Asst. Engr., Board of Water Supply, New York City, in charge of about 1 1/2 miles of Catskill Aqueduct at Kitchawan, N. Y. *TT 0.9: P 0.9: RC 0.9.*—April 1910 to Jan. 1917 Constr. Engr., E. E. Smith Contr. Co., on 4-track subway, 4th Ave. (26th to 43d St.), Brooklyn, and Broadway (16th to 27th St.), New York City, and foundations for concrete viaduct, Queens Boulevard, Long Island City, having charge of all construction and cost records. *TT 6.7: P 6.7: RC 6.7.*—July 1922 to Aug. 1923 Supt., until Sept. 1922 for Nelson-Pedley Co., Philadelphia, in charge of foundation work for 10-story building and shoring and underpinning adjacent properties, then with Keystone State Constr. Co., in charge of contractor's forces, constructing river piers and caisson work for land anchorage on Philadelphia side for Philadelphia-Camden Suspension Bridge. *TT 1.1: P 1.1: RC 1.1.*—Oct. 1923 to Feb. 1924 Engr. Supt., James Stewart Co., Inc., Chicago, Ill., supervising driving piles and placing concrete foundations for grain elevator in Philadelphia, and pneumatic caisson foundations for grade elevator at Oswego, N. Y. *TT 0.3: P 0.3: RC 0.3.*—Jan. 1917 to July 1922 and Feb. 1924 to Jan. 1931 Div. Engr., and Jan. 1931 to date Constr. Engr., Dept. of City Transit, Philadelphia, in charge of construction of 4-track subway station beneath City Hall and 2-track subway on Arch St. (8th St. to Broad St.) (5 1/2 years) and of 10 000-ft. section of 4-track subway, North Broad St., all track construction, large part of station-finish work, construction of 2-track subway and concourse under City Hall (about 7 years), and since Jan. 1931 in charge of all construction of Dept., about 5 miles of 2-track subway construction, including track laying, station finish and control, etc. (total about $30 000 000). *TT 13.3: P 13.3: RC 13.3.*—*TT 31.5: SP 1: P 30.5: RC 25.3.* Refers to L. Costello, G. S. Frost, S. Harris, J. D. Jaques, L. B. Manley, W. R. Scanlin, R. C. Scott, C. H. Stevens, S. M. Swaab.

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(9) **SOURS, HAROLD GLENN,** County Engr.'s Office, Akron, Ohio. (Age 37. Born Manchester, Ohio.) 1916 B. Sc., Univ. of Akron. *TT 2: P 2.*—June 1916 to June 1918 Chf. of Party and Office Designer, Barstow & McCurdy, Akron, Ohio, surveys, staking out subdivisions, establishing street lines and lot corners, platting and computing allotment plats, planning and estimating street grading, sidewalks and sewers, supervising construction, etc. *TT 1.5: SP 0.5: P 1.*—July to Dec. 1918 with U. S. Army, Ft. Monroe, Va., 3 months in Coast Artillery School, commissioned 2nd Lieut., Coast Artillery. *TT 0.3: SP 0.1: P 0.2.*—Jan. 1919 to date with Ohio State Highway Dept., in Summit County, until June 1920 as Maintenance Supt., in charge of maintenance and repair, then Constr. Engr., in charge of construction, surveys, plans and estimates, and since Sept. 1921 Res. Engr. in charge of county highway, bridge and drainage work, including surveys, plans, estimates, specifications and construction, in charge of maintenance and repair of county highways, since Sept. 1925 also County Engr. of Summit County. *TT 13: P 13: RC 10.3.*— *TT 16.8: SP 0.6: P 16.2: RC 10.3.* Refers to M. L. Davis, C. E. Grubb, G. F. Pfeiffer, G. B. Sowers, F. E. Swineford, E. A. Zeisloft.

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(12) **STEWART, ROBERT SHORTRIDGE,** 2275 East Sixtieth St., Seattle Wash. (Age 29. Born Grand Forks, N. Dak.) 1931 B. S. in C. E., Univ. of Wash. *TT 4: p 4.*—Jan. 1924 to Nov. 1925 Cost Clerk, Pacific Gas & Elec. Corporation, San Francisco, Cal., in construction camp for large hydro-electric project. *TT 0.9: SP 0.9.*—June to Sept. 1928 Inspector, City of Seattle, Wash., on paving. *TT 0.1: SP 0.1.*—Sept. 1928 to Jan. 1929 Member of firm and Gen. Foreman, Stewart & Breen, Seattle, paving, in charge of work. *TT 0.3: P 0.3: RC 0.3.*—Jan. to Oct. 1929 Sales Engr., B. F. Easterbrooks Co., Seattle, in charge of all engineering and layout of material-handling systems. *TT 0.8: P 0.8: RC 0.8: D 0.8.*—June 1930 to Jan. 1931 Sales Engr., Consolidated Steel Corporation, Los Angeles, being Crane Engr. and Salesman for western Washington. *TT 0.5: P 0.5: RC 0.5*— Summer 1931 Chf. of Party, Dept. of Interior, National Park Service, on layout and supervision of construction of roads, trails, pump and power houses, water and transmission lines.—*TT 6.6: SP 1: P 5.6: RC 1.6: D 0.8.* Refers to G. E. Hawthorn, C. C. More, R. G. Tyler.

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(11) SULLIVAN, CLARENCE THIEL, 6339 West Sixth St., Los Angeles, Cal. (Age 33. Born Portland, Ore.) 1923 B. S. C. E., Ore. Inst. Tech. *TT 4: P 4*—Sept. 1919 to May 1920 Chainman and Rodman, Pacific Spruce Corporation, on railroad construction. *TT 0.3: SP 0.3*—June to Sept. 1920 Chainman, Rodman and Levelman, Long Bell Lumber Co., on mapping and surveys. *TT 0.2: SP 0.2*—June 1923 to March 1924 Prin. Asst., Govt. Bridge Dept., on construction of Rogue River Suspension Bridge, 5 months (in absence of superior) in charge of work. *TT 0.8: P 0.8: RC 0.4*—April 1924 to Oct. 1925 Field Engr., Gen. Petroleum Corporation, Los Angeles, Cal., having responsibility in connection with design and construction of plants in Seattle, Wash., Portland, Ore., and San Francisco and Oakland, Cal. *TT 1.5: P 1.5: RC 1: D 1*—Oct. 1925 to Jan. 1926 Engr., Johns-Manville, Inc., on construction of Pacific Coast Plant, Pittsburg, Cal., being responsible for design of foundations (building and machinery) and structural steel work. *TT 0.3: P 0.3: RC 0.3: D 0.3*—Feb. 1926 to date with Union Oil Co., Los Angeles, until Aug. 1926 as Asst. Res. Engr., in charge of construction in Oregon (2 months) and of design and construction of bulk stations, marine terminals and refineries in Alaska and Canada (4 months); Aug. 1926 to June 1927 Asst. to Constr. Engr. on special assignments, consisting of design and construction of reservoirs, fire-walls, compressor plants, etc.; June to Dec. 1927 Asst. Res. Engr., first in charge of construction of bulk storage plant and marine terminal at Hoquiam, Wash., designed (in field) and built fire-walls, docks, and heavy pile foundations with reinforced concrete pads for tankage, and after Oct. in charge of design and preliminary construction of reinforced concrete drain on piling, in Portland; Dec. 1927 to Nov. 1928 Res. Engr. in charge of all engineering work for distribution group on Pacific Coast reporting to Constr. Engr., since Nov. 1928 Cons. Engr. in charge of all design and construction for same group. *TT 5.7: P 5.7: RC 5.7: D 5.1*—*TT 12.8: SP 0.5: P 12.3: RC 7.4: D 6.4*. Refers to F. W. Karge, G. S. Lane, R. R. Moyer, R. J. Reed, W. E. Whittier

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(16) VICKERY, ALBION KENT, 2088 Forest St., Denver, Colo. (Age 60. Born in Essex County, N. Y.) 1889 to 1892 and 1893 to 1898 with City and County of Denver as Rodman, Chainman, Transitman and (3 years) Asst. Engr. in charge of field work. *TT 5.5: SP 2.5: P 3*.—1892 to 1893 Transitman on irrigation project in Wyoming. *TT 0.5: SP 0.5*.—1898 to 1900 Location Engr., Northwest Irrigation Co., on irrigation project in Alberta, Canada. *TT 2.5: P 2.5: D 2.5*.—1900 to 1901 Asst. Location Engr., Colorado Fuel & Iron Co. *TT 1.5: P 1.5: D 1.5*.—1901 to 1904 Chf. Deputy City Engr., Denver, in charge of surveys and of some construction. *TT 3: P 3: D 3*.—1904 to 1919 Cons. and Contr. Engr., Denver, 2 years as member of firm, Vickery, Foster & Collins, and remainder of time of firm Vickery, Foster & Doll, on surveys, designing plans for irrigation projects, sewer and water-works systems for small towns, some contracting (approx. $300 000), including construction of small railroad bridges, concrete construction on dam and tunnel at Valier, Mont., grading, curb and gutter construction and some storm-sewer work for City and County of Denver, some water and sewer construction at various places. *TT 15: P 15: RC 15: D 15*.—1919 to date with City and County of Denver until 1921 as Gen. Supt. of Parks, in charge of road and bridge construction in mountain parks and of improvements in city park systems, and since 1921 Chf. Engr. in full charge of surveys, establishing grades, construction of pavements and of storm and sanitary sewers, widening streets, etc. (total approx. $25,000,000). *TT 13: P 13: RC 13: D 3*—*TT 41: SP 3: P 38: RC 28: D 25*. Refers to J. B. Bertrand, H. S. Crocker, W. W. Curtis, W. B. Freeman, D. D. Gross, M. C. Hinderlider, V. K. Jones, A. O. Ridgway, C. D. Vail.

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(1) VIRGILIO, NICHOLAS, 4133 Paulding Ave., New York City. (Age 23. Born New York City.) 1931 B. S. in C. E., Manhattan Coll. *TT 4: P 4*.—Jan. to Sept. 1927 Eng. Asst., Leo J. Ehrhart, Inc., Engrs. and Contrs., New York City, acting as Chainman, Rodman, Notekeeper, Computer, Draftsman, etc. *TT 0.3: SP 0.3*.—Summers 1929 and 1930 successively, Jun. Draftsman, Consolidated Tel. & Elec. Subway Co., New York City, on drawings, etc. and Topographical Draftsman with Harry J. Philips, Engr., Pelham Manor, N. Y., computing, chaining, acting as Rodman, Transitman, etc.—*TT 4.3: SP 0.3: P 4*. Refers to J. J. Costa, J. A. Ruddy.

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(11) VON KARMAN, THEODOR, California Inst. of Technology, Pasadena, Cal. (Age 50. Born Budapest, Hungary.) 1902 M. E. Kiralyi Jozsef Muegmetem (Tech. Univ.), Budapest, Hungary. 1908 Ph. D., Univ. of Goettingen, Germany. D. E. (Hon.) Tech. Hochschule, Berlin. *TT 4: P 4*.—1902 to 1903 Private, Austrian-Hungarian Army.—1903

to 1906 Research Engr., Ganz & Co. Machine Works, Budapest. *TT 1.5: SP 1.5.*—1906 to 1908 graduate student, and 1908 to 1912 Instructor, Univ. of Goettingen. *TT 4: P 4.*—1912 to 1914 and 1918 to 1930 Prof., Univ. of Aachen; Director of Aeronautical Inst. *TT 14: P 14: RC 14.*—1914 to 1918 with Austrian Hungarian Army, as Lieut., Capt., and Chf. of Experimental Dept., Aviation Service. *TT 4: P 4: RC 4: D 2.*—1930 to date Prof., California Inst. of Technology; Director of Aeronautical Laboratory. *TT 1.5: P 1.5: RC 1.5.*—*TT 29: SP 1.5: P 27.5: RC 19.5: D 2.* Refers to F. J. Converse, R. R. Martel, W. W. Michael, F. Thomas, H. M. Westergaard.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

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(15) ARNESON, EDWIN PERCIVAL, Assoc. M., 418 Gunter Office Bldg., San Antonio, Tex. (Elected May 12, 1919.) (Age 43. Born Ft. Worth, Tex.) 1910 B. Sc. in Civ. Eng., and 1919 C. E., Agri. & Mech. Coll. of Tex. *TT 4: P 4.*—Nov. 1906 to Sept. 1907 Rodman and Draftsman, Ft. Worth & Denver City Ry. *TT 0.4: SP 0.4.*—June 1910 to July 1911 Structural Draftsman, Eng. Dept., The Texas Co., Houston, Tex., on design of warehouses, buildings, wharves, steel tanks, etc. *TT 1.2: P 1.2: D 1.2.*—Aug. 1911 to May 1913 Office Engr., Medina Irrigation Co., San Antonio, Tex., on surveys and construction of canal system, had charge of estimates, cost keeping and supervised design of canal structures, siphons, flumes, etc., and acted in executive capacity. *TT 1.8: P 1.8: RC 1.8: D 1.8.*—June 1913 to Sept. 1914 Asst. Engr., on various projects, Ebro Irrigation & Power Co., Barcelona, Spain, in responsible charge of surveys, irrigation and hydro-electric projects, estimates and designs. *TT 1.3: P 1.3: RC 1.3: D 1.3.*—Dec. 1914 to May 1915 Engr. in immediate charge of surveys and studies for drainage project, Waco, Tex. *TT 0.5: P 0.5: RC 0.5: D 0.5.*—June 1915 to July 1916 Chf. Draftsman, Office of City Engr., San Antonio, Tex., supervised design of miscellaneous structures and estimates, extensive municipal improvements. *TT 1.2: P 1.2: RC 1.2: D 1.2.*—Aug. to Nov. 1916 Engr. in Dept. Engr.'s Office Southern Dept., U. S. Army, Ft. Sam Houston, San Antonio, Tex., designing warehouses, hospitals, office, etc. *TT 0.3: P 0.3: D 0.3.*—June 1918 to April 1919 Structural Draftsman, Navy Dept., Bureau of Yards and Docks, Washington, D. C., on design of shipbuilding ways, crane runways and water-front improvements. *TT 0.8: P 0.8: D 0.8.*—Dec. 1916 to June 1918 and May 1919 to March 1930 member of firm, Walter & Arneson, Civ. Engrs., San Antonio, Tex., on topographic surveying, irrigation and highway projects, irrigation studies, structural designing and general civil engineering. *TT 11.3: P 11.3: RC 11.3: D 3.3.*—March 1930 to date in private practice of civil engineering, mainly highway engineering. *TT 1.7: P 1.7: RC 1.7: D. 0.7.*—*TT 24.5: SP 0.4: P 24.1: RC 17.8: D 11.1.* Refers to G. Gilchrist, J. L. Lochridge, A. J. McKenzie, J. T. L. McNew, J. A. Norris, E. W. Robinson, A. P. Rollins, J. A. Sargent, W. O. Washington.

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(7) BROWN, HERBERT HENRY, Assoc. M., 2959 North Thirty-sixth St., Milwaukee, Wis. (Elected Oct. 1, 1928.) (Age 37. Born Milwaukee, Wis.) 1917 B. S., Univ. of Wis. *TT 4: P 4.*—Feb. 1913 to May 1917 (while student) on mechanical drawing and time-keeping, Cutler Hammer Mfg. Co. (about 7 months), laying reinforcing steel, Newton Eng. Co. (3 months), and shop work, Chicago, Milwaukee & St. Paul Ry. Co. (3 months).—May 1917 to May 1919 Capt. of Infantry, U. S. Army, on construction of large target range, complete set of field fortifications, special surveys, construction, moving and razing of buildings, etc. *TT 1: P 1.*—June 1919 to June 1920 Estimator, Draftsman and Designer, Worden Allen Co., Milwaukee, on structural steel work. *TT 1: P 1: D 0.5.*—June 1920 to Jan. 1922 Mech. Draftsman, Prime Mfg. Co., Milwaukee. *TT 1.5: P 1.5.*—Jan. 1922 to date with City of Milwaukee as follows: Jan. to June 1922 Draftsman on plans for new pumping station; June 1922 to May 1923 Asst. Engr., in charge of grade separation, design of grade separation projects and laying out work in field; May 1923 to April 1924 Structural Designer, design and drawing work on Riverside Pumping Station; April 1924 to March 1926 Gen. Eng. Designer, surveys, plans, specifications and designs for retaining walls, hollow sidewalk reconstruction, grade separation projects, etc.; March 1926 to Jan. 1927 Special Asst. Engr., on special surveys, reports and investigations on various projects for City Engr.; since Jan. 1927 Engr. in charge of design and construction of pumping stations, being responsible for design, installation of mechanical equipment and construction of buildings for Water Dept., including pumping stations, elevated steel tanks, etc. *TT 10: P 10: RC 5: D 2.* *TT 17.5: P 17.5: RC 5: D 2.5.* Refers to R. H. Cahill, T. C. Hatton, E. L. Knebes, R. R. Lundahl, J. P. Schwada, C. U. Smith.

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(4) **COURTENAY, WILLIAM ALOYSIUS, Jr.**, Assoc. M., 23 Parkway Ave., Chester, Pa. (Elected Oct. 14, 1919.) (Age 48. Born Philadelphia, Pa.) 1900 to 1906 with William Cramp & Sons Ship & Engine Bldg. Co., designing and superintending erection. *TT 3.5: SP 2.5: P 1: RC 1.*—1906 to 1907 with Southeastern Constr. Co., drafting and field work on Philadelphia & Western R. R. *TT 0.5: SP 0.5.*—1907 to date with Sun Oil Co., until 1920 as Plant Engr. at Marcus Hook, Pa., in charge of design and erection of work for oil refinery, and since 1920 Mgr., Constr. Dept., Philadelphia, in charge of design and erection of service stations and bulk terminal facilities, duties included charge of design, estimates, field construction forces, supervision of drafting room, etc.; since 1914 also Borough Engr. of Marcus Hook. *TT 25: P 25: RC 22: D 3.*—*TT 29: SP 3: P 26: RC 23: D 3.* Refers to J. Adler, A. G. Chapman, H. S. Farquhar, V. Gelineau, W. R. Hughes, Jr., A. C. Kaestner, E. Kitchen, W. H. Nason, J. H. Neeson, H. J. Sherman, J. G. Shryock, C. R. Simpson, B. White.

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(2) **CURRY, TRUMAN MINOR, Jr.**, Assoc. M., Box 102, Watertown, Conn. (Elected Feb. 10, 1930.) (Age 37. Born New York City.) 1916 C. E., Columbia Univ. *TT 4: P 4.*—June 1916 to May 1917 Asst., Eng. Dept., New York Central R. R., Poughkeepsie, N. Y., acting as Asst. Inspector on construction of railroad station (½ year) and on miscellaneous drafting, surveying and engineering investigations. *TT 0.6: SP 0.3: P 0.3.*—July 1917 to Jan. 1920 with Corps of Engrs., U. S. Army, about 4 months as Lieut., then Capt., in responsible charge, acting as Instructor, Bridge Section, Army Engr. School, Langres, France, prepared course of lectures and supervised construction and field work (Feb.–Aug. 1918); Company Commander, 6th U. S. Engrs., A. E. F. (2 months); Camp Utilities Officer, Camp Upton, N. Y. (4 months), and remainder of time as student in various army schools. *TT 2.5: P 2.5: RC 1.1.*—Feb. 1920 to June 1926 Engr. with Hugh L. Thompson, Cons. Engr., Waterbury, Conn., on engineering work for industrial plants, etc., supervising and expediting construction, preparing specifications for and ordering materials, equipment and labor, and designing structures. *TT 6.4: P 6.4: RC 3.2: D 1.6.*—July 1926 to Jan. 1930 Asst. Engr., Bureau of Eng., Waterbury, Conn., in charge of Trunk Sewer Dept., directed plans and specifications, arranged for contracts and supervised construction of new intercepting sewer system. *TT 3.5: P 3.5: RC 3.5: D 1.*—Feb. 1930 to date Associate Engr. with Nicholas S. Hill, Jr., Cons. Engr., New York City, in charge of construction and specifications for water-purification and sewage-treatment plants, sewers, etc., also conducted investigations and made reports proposing new construction or improvements. *TT 1.9: P 1.9: RC 1.9.*—*TT 18.9: SP 0.3: P 18.6: RC 9.7: D 2.6.* Refers to A. N. Aeryns, R. A. Cairns, H. H. Chase, E. W. Clarke, J. K. Finch, N. S. Hill, Jr., S. K. Knox.

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(9) **DAUDT, RALPH BRUERE**, Assoc. M., 2629 Robinwood Ave., Toledo, Ohio. (Elected May 7, 1913.) (Age 44. Born Toledo, Ohio.) 1907 A. B., and 1910 C. E., Cornell Univ. *TT 4: P 4.*—June 1909 to date with The A. Bentley & Sons Co., Toledo, Ohio, until Jan. 1910 as Asst. Field Engr. on construction of dry-dock for Toledo Shipbuilding Co.; Jan. 1910 to Jan. 1911 Field Engr. on construction of Kinsey Mfg. Co.'s building (2 months) and of Woolson Spice Co.'s building (3 months), both in Toledo, and of Evansville (Ind.) Filter Plant (7 months), also Asst. Supt. on the latter; Jan. to July 1911 Asst. Supt. on construction of Beaver Power Bldg. and Schwind Bldg., Dayton, Ohio; July to Oct. 1911 Branch Mgr., Dayton Office; since Oct. 1911 Chf. Engr. in charge of estimating and technical purchases, also of design and construction of power, commercial, industrial, water and sewage plants, docks, bridges, etc.; since July 1922 also Vice-Pres. *TT 22.2: P 22.2: RC 20.2: D 20.*—*TT 26.2: P 26.2: RC 20.2: D 20.* Refers to L. M. Gram, H. P. Jones, G. V. Rhines, G. N. Schoonmaker, G. A. Taylor.

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(1) **GENTHON, RENE MARIUS**, Assoc. M., 157 Second St., Clifton, N. J. (Elected Jan. 14, 1924.) (Age 38. Born Clifton, N. J.) 1915 B. S. in Civ. Eng., and 1923 C. E., Manhattan Coll. *TT 4: P 4.*—1 year (prior to June 1915) surveying on various projects.—June 1915 to April 1916 on surveys and developments, and (4 months) on highway construction under County Engr. of Hudson County, N. J. *TT 0.9: P 0.9: RC 0.5: D. 0.5.*—April 1916 to April 1917 with J. H. Grozier Co., Contrs., Hartford, Conn., as Estimator and Detailer of industrial construction, later Asst. Supt., Clerk of Works. *TT 1: P 1: RC 1: D 0.8.*—April 1917 to Oct. 1919 with U. S. Army, 1st Engrs., 1st Div., as Private, etc., and finally (1918) Lieut., 2 years overseas; had charge of surveys and highway location at various camps, also cantonment construction, water supply and sanitation;

after 1918 Reg. Constr. Officer on battle front, railroad camp sanitation and mining operations, later Instructor in Military bridge Eng., design, construction and demolition at Third Corps School for Officers (A.E.F.), and San. Engr. with Army of Occupation in Germany. *TT 2.5: P 2.5: RC 2.5: D 2.*—Oct. 1919 to April 1920 surveys, development, street layouts, grade maps, drainage, etc., estimating and designing for local contractors. *TT 0.5: P 0.5: RC 0.5: D 0.5*—April 1920 to July 1921 with Passaic Water Co., Montclair Water Co., and eleven allied companies, being Engr. in charge of construction, including 3 miles of 30-in. lock bar steel pipes through Paterson, N. J., etc.; Asst. to Chf. Engr. on valuation reports for properties of East Jersey Water Co. and allied companies and design of additions in charge of design of reservoir with distribution system (16 miles, 4-to 24-in. pipe) for Haledon Borough, N. J., study of distribution system and report of recommendation for Overbrook and Caldwell Institutions of Essex County N. J. *TT 1.2: P 1.2: RC 1.2: D 1.2.*—July 1921 to Jan. 1928 Asst. City Engr., Clifton, in charge of surveys, monumentation, grade maps, establishing grades, estimates, specifications, records and reports, design of storm-water sewers and drains ($120 000), of additions to water-distribution system, and (under direct co-operation of City Engr.) of studies for grade crossing elimination and park development, layout of new streets (paving program, estimated $2 000 000); assisted in design and supervision of construction of sanitary sewer system ($2 500 000); designed and supervised construction of fire- and police-alarm systems; made study of traffic for police regulation, etc. *TT 6.6: P 6.6: RC 6.6: D 6.6.*—Jan. 1928 to Oct. 1929 estimating, detailing and cost accounting on sewers, waterworks, bridges, etc., for numerous contractors and engineers, including principally work as Res. Engr. for Watson & Pugh, Cons. Engrs. of New York and Philadelphia, on appraisal and studies for improvement and extension of sewerage system in Long Branch, N. J., design of extensions and improvements to sewerage system of Bloomfield, N. J., for City Engr., small bridges, culverts, etc. *TT 1.7: P 1.7: RC 1.7: D 1.7.*—Oct. 1929 to date Res. Engr. for Geo. A. Johnson, Cons. Engr., New York City, assisted in design and was Res. Engr. on construction of sewage-disposal plant, with booster stations, clarifiers, separate sludge digesters, sludge drying beds, etc. (total $1 000 000), and sewerage system (trunk sewers up to 54 in., total about $900 000) for Norwalk, Conn., made detailed designs of various parts and had charge of construction, inspections, changes in design, estimates, etc. *TT 2.1: P 2.1: RC 2.1: D 2.1.*—*TT 20.5: P. 20.5: RC 16.1: D 15.4.* Refers to F. S. Childs, A. W. Cuddeback, G. Ferguson, G. A. Johnson, E. A. MacMillan, A. S. Mahony, G. L. Watson.

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(15) HUFFMAN, THOMAS ELLSWORTH, Assoc. M., 1203 Nixon Bldg., Corpus Christi, Tex. (Elected Aug. 4, 1924.) (Age 50. Born Antioch, Ohio.) 1907 C. E., Ohio Northern Univ. *TT 4: P 4.*—1907 to 1910 on railroad location and construction, until 1908 as Transitman, then Res. Engr. for Artesian Belt Ry. Co. on construction. *TT 2.2: SP 0.5: P 1.7.*—1911 to 1913 Chf. of Party on railroad location and irrigation surveys, estimates, etc., under San Antonio (Tex.) Chamber of Commerce, etc. *TT 1.7: P 1.7: RC 1.*—1913 to 1915 Topographer and Draftsman, St. Louis & South Western Ry. Co. *TT 1: SP 1.*—1915 to 1922 in private practice of highway engineering, making surveys, plans and specifications and supervising construction, including about 40 miles of highway for Cooke County, Tex., and, as Engr. for Denton County, a road-building program ($2 000 000), involving approx. 200 miles of highway. *TT 7: P 7: RC 7: D 7.*—1922-1923 made study of highway problems in various states; also special student at Massachusetts Inst. of Technology.—Nov. 1923 to March 1930 County Engr., Webb County, Tex., in charge of surveys, plans and construction on State and County highways. *TT 6.3: P 6.3: RC 6.3: D 6.3.*—Aug. to Nov. 1923 and March 1930 to date with Texas State Highway Dept., first as Div. Engr. in charge of construction and maintenance, and since March 1930 Res. Engr. in charge of surveys, plans and construction. *TT 2: P 2: RC 2: D 1.7.*—*TT 24.2: SP 1.5: P 22.7: RC 16.3: D 15.* Refers to T. W. Bailey, S. F. Crecelius, C. M. Davis, J. M. Howe, O. H. Koch, J. T. L. McNew.

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(16) JONES, CLARENCE STRAIN, Assoc. M., 312 Vine St., Leavenworth, Kans. (Elected Junior Sept. 2, 1914; Assoc. M. April 19, 1920.) (Age 41. Born Leavenworth, Kans.) Sept. 1909 to June 1911 Student, School of Eng., Kansas Univ. *TT 1: P 1.*—Aug. 1912 to April 1913 with Kansas City (Mo.) Bridge Co., as Asst. Engr. and Timekeeper on West Kansas Ave. bridge. *TT 0.3: SP 0.3.*—May to Oct. 1914 Draftsman, Kansas City Structural Steel Co., detailing mill buildings. *TT 0.2: SP 0.2.*—Oct. 1914 to Jan. 1916 Engr., Monarch Eng. Co., Falls City, Nebr., in charge of construction of highway bridges in South Dakota, Nebraska and Oklahoma. *TT 1.2: P 1.2: RC 1.2.*—Jan. 1916 to May 1917 member of firm, Jones & Barth, Cons. Engrs., bridge and building work, etc. *TT 1.3:*

P 1.3: RC 1.3: D 0.5.—June 1911 to Aug. 1912, April 1913 to May 1914 and May 1917 to date with Missouri Valley Bridge and Iron Co., first as Templet Maker and Draftsman, and after April 1913 Asst. Engr. designing and detailing superstructures, substructures and falsework for bridges, etc.; about 2 months loaned to Hedrick & Cochrane, Cons. Engrs., Kansas City, designing and detailing part of machinery for Pine Bluff Lift Bridge. *TT 15.5: SP 1.2: P 14.3: RC 13.6: D 14.1.*—*TT 19.5: SP 1.8: P 17.7: RC 16.2: D 14.6.* Refers to E. H. Barkmann, V. H. Cochrane, E. H. Connor, I. G. Hedrick, H. C. Tammen, H. P. Treadway, H. S. Tullock, F. E. Washburn, O. A. Zimmerman.

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(12) KIMBRELL, GEARY, Assoc. M., 544 East Fifty-seventh St. North, Portland, Ore. (Elected June 16, 1919.) (Age 54. Born Union, Ore.) Sept. 1895 to June 1899 student, Univ. of Oregon. *TT 2: P 2.*—June 1899 to July 1902 successively Transitman (4 months), Topographical Draftsman (9 months), Supt. of Constr. for Pendleton (Ore.) Water Works (4 months) and Architectural Draftsman (about 1¾ years). *TT 3: P 3: RC 0.3: D 1.2.*—July 1902 to June 1919 with Umatilla County, until Jan. 1911 as Deputy County Surveyor, then County Surveyor, on surveys and highway location, and after Jan. 1915 County Highway Engr. in charge of location, design and construction of highways and highway structures, including steel and reinforced concrete bridges, rock walls, rock grading and surfacing; also City Engr. of Pendleton, on design and construction of sewer system ($50 000), street improvements (over $620 000), a reinforced concrete and two steel highway bridges, and reinforced concrete retaining walls (special design); also on private work, including surveys, designs, estimates, reports, etc., on city and townsite work and for irrigation and power companies, also technical witness, etc., design and construction of levee ($26 000) for Eastern Oregon State Hospital, survey, design and estimate for hydro-electric power plant for Milton, Ore., etc. (given in detail in application). *TT 16.5: P 16.5: RC 11.1: D 6.1.*—June to Aug. 1919 Highway Engr., U. S. Govt. Bureau of Public Roads, in charge of location of 50-mile project across Wasatch Mountains, Utah. *TT 0.2: P 0.2: RC 0.2.*—Sept. 1919 to date with City of Portland, Ore., until Dec. 1922 as Draftsman, Public Works Dept., on design and estimates of sewers and street improvements, street extension, evaluation and reports, then Draftsman, Bridge Div., on designs and estimates, and since March 1925 Asst. Bridge and Highway Engr. on design and estimates of bridges, retaining walls and water-front seawall. *TT 12.2: P 12.2: RC 8: D 10.5.*—*TT 33.9: P 33.9: RC 19.6: D 17.8.* Refers to J. W. Cunningham, F. T. Fowler, O. Laurgaard, C. B. McCullough, H. A. Rands, C. W. Raynor, O. E. Stanley.

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(1) PETTIGREW, ROBERT LESLIE, Assoc., M., Cape Haitien, Haiti. (Elected June 19, 1922.) (Age 38. Born Salem, Mo.) 1915 B. S. in C. E., Mont. State Coll. *TT 4: P 4.*—June 1915 to June 1917 Asst. Engr. in Forest Products, U. S. Forest Products Laboratory, Madison, Wis., tests on structural woods and research on forest products. *TT 2: P 2.*—Aug. 1917 to Feb. 1927 Commissioned Officer, Corps of Civ. Engrs., U. S. Navy 6 months as Chf. Draftsman and Senior Asst. to Public Works Officers, Navy Yard, New York, on drafting, design and specifications; 6 months special work on specifications and tests of wood parts for aircraft construction, Washington, D. C.; 6 months in charge of construction, Naval Air Station, Brunswick, Ga.; 2 5/6 years Public Works Officer, Naval Station, Virgin Islands, also in charge of Public Works for Island Govt., including design and construction of general municipal works; 1 1/3 years Senior Asst. to Public Works Officer, Norfolk, (Va.) Navy Yard; 3 5/6 years Asst. to Engr. in Chf., Republic of Haiti, on design and construction of general municipal and government projects, including waterworks, drainage, roads, irrigation and public buildings. *TT 9.5: P 9.5: RC 9.5: D 3.*—Feb. 1927 to date Mgr., Haitian American Development Corporation, in charge of development of agricultural and manufacturing enterprise, including selection, design and construction of railroad, power plant, water supply, shipping facilities, factory and buildings, transmission line, etc. *TT 4.8: P 4.8: RC 4.8: D 2.*—*TT 20.3: P 20.3: RC 14.3. D 5.* Refers to F. H. Cooke, L. E. Gregory, F. C. Nyland, A. L. Parsons, D. C. Webb.

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(2) WILLIAMS, HOWARD JAMES, Assoc. M., 44 School St., Boston, Mass. (Elected Oct. 10, 1927.) (Age 36. Born Kingston, Ont., Canada.) 1917 B. S., Queens Univ. 1920 M. S., Mass. Inst. Tech. *TT 4: P 4.*—April to Nov. 1917 Field Engr., Cedar Rapids Power Co., Cedars, Que., on construction. *TT 0.3: SP 0.3.*—Nov. 1917 to Oct. 1919 with Hydro Elec. Power Comm. of Ontario, on Chippewa-Queenstown Power Development as Field Engr. on construction. *TT 1.6: SP 0.3: P 1.3.*—Oct. 1919 to June 1920 Graduate student, Massachusetts Inst. of Technology.—June 1920 to Feb. 1923 Asst. Engr., Maine

Water Power Comm., Augusta, Me., investigating and reporting on water-power resources of the State. *TT 2.7: P 2.7: RC 2.7: D 1.*—March to April 1923 Asst. Engr. with John F. Vaughan, Cons. Engr., Boston, on stream-flow and storage studies. *TT 0.1: P 0.1.*—April 1923 to July 1926 Asst. Engr., Designing Div., Water-Supply Board, Providence, R. I., on design of structures for Scituate Reservoir, aqueduct, distribution reservoirs and mains, valuation of condemned mills and water powers. *TT 3.2: P 3.2: RC 2.2: D 2.2.*—July 1926 to date Asst. Engr., with Fay, Spofford & Thorndike, Cons. Engrs., Boston, Mass., being Prin. Hydr. and Structural Designer on several dams, reservoirs, water-supply and sewerage systems, bridges and buildings, preparing plans, specifications and reports. *TT 5.5: P 5.5: RC 5: D 5.*—*TT 17.4: SP 0.6: P 16.8: RC 9.9: 8.2.* Refers to H. K. Barrows, A. C. D. Blanchard, C. A. Farwell, F. H. Fay, C. M. Spofford, F. E. Winsor.

FROM THE GRADE OF JUNIOR

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(15) **DE GEURIN, LEWIS CLAUDIUS,** Jun., Overton, Tex. (Elected Aug. 18, 1930.) (Age 27. Born Overton, Tex.) 1926 B. S. in Civ. Eng., La. State Univ. *TT 4: P 4.*—Oct. 1921 to May 1922 Jun. Architectural Draftsman, Office of Superv. Archt. Treasury Dept., Washington, D. C. *TT 0.3: SP 0.3.*—Feb. to Sept. 1924 Transitman for R. H. Parkinson, Engr., San Antonio, Tex. *TT 0.3: SP 0.3.*—Jan. to May 1925 Draftsman, Louisiana State Highway Dept., Baton Rouge, La., plotting level notes, quantity surveys, etc.—July to Sept. 1926 Instructor in Civ. Eng. for L. Mondel (private school) and Structural Draftsman, Union Carbide & Carbon Co., both of New York City. *TT 0.1: SP 0.1.*—Sept. 1926 to date in private practice, until Sept. 1931 as Civ. Engr. at Harlingen, Tex., and Phoenix, Ariz., on designs of steel and concrete structures, etc., and since then as Land Surveyor and Cons. Engr., at Overton, Tex. *TT 5.3: P 5.3: RC 5.3: D 5.3.*—*TT 10: SP 0.7: P 9.3: RC 5.3: D 5.3.* Refers to W. H. Becker, J. Leahy, B. W. Pegues, A. Tamm, P. B. Tartt.

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(4) **FRAIVILLIG, LEONARD MARTIN,** Jun., 728 Center St., Bethlehem, Pa. (Elected Oct. 1, 1926.) (Age 27. Born Schenectady, N. Y.) 1926 C. E., Lehigh Univ. *TT 4: P 4.*—June 1926 to Aug. 1928 with Dravo Contr. Co., Pittsburgh, Pa., until Oct. 1927 as Estimator and Designer, on bridges, dams, walls, cofferdams and caissons, then Res. Engr., on construction of U. S. Dam No. 6, Allegheny River, in charge of engineering work, monthly estimates and progress reports. *TT 2.1: P 2.1: RC 0.8: D 0.3.*—Aug. to Nov. 1928 Designer for F R. Harris, Cons. Engr., New York City, on sheet pile sea-walls and reinforced concrete and structural steel. *TT 0.3: P 0.3: D 0.3.*—Nov. 1928 to date Chf. Draftsman and Designer, Eng. Dept., City of Bethlehem, Pa., on municipal works and projects. *TT 3: P 3: RC 3: D 3.*—*TT 9.4: P 9.4: RC 3.8: D 3.6.* Refers to L. P. Bailey, R. J. Fogg, R. L. Fox, H. G. Payrow, W. L. Wilson.

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(11) **GARDNER, RUSKIN TENNEY,** Jun., 323 West Vernon St., Phoenix, Ariz. (Elected April 18, 1927.) (Age 28. Born Thatcher, Ariz.) 1926 B. S. in C. E., Univ. of Ariz. *TT 4: P 4.*—June 1926 to April 1927 Draftsman, Middle Rio Grande Conservancy Dist., Albuquerque, N. Mex., drafting estimating quantities and checking design computations on irrigation, drainage and flood-protection project. *TT 0.5: SP 0.5.*—April 1927 to June 1928 Draftsman, Inspector and Asst. Designer, Maricopa County Municipal Water Conservation Dist. No. 1, Phoenix, Ariz., drafting, designing and estimating quantities for irrigation structures and Field Inspector on installation of irrigation system and structures. *TT 0.5: SP 0.5.*—June 1928 to date with City Water Dept., Phoenix, as Engr. assisting City Mgr. and special consultants in preparing preliminary reports recommending major expansion of water and sewer system; at present Asst. Supt. of Water Dept., supervising operation of Dept. *TT 3.5: P 3.5: RC 2.5: D 1.*—*TT 8.5: SP 1: P 7.5: RC 2.5: D 1.* Refers to J. A. Fraps, C. E. Griggs, A. F. Harter, C. H. Howell, F. C. Kelton, A. W. Newcomer, C. A. Smith.

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(14) **HIGHFILL, JAMES BODENHEIMER,** Jun., Rex-Moore Apts., Seminole, Okla. (Elected March 11, 1929.) (Age 29. Born Trinidad, Colo.) Student, School of Eng., Milwaukee, Wis. (Sept. 1918 to June 1919), and in Civ. Eng., Univ. of Wis. (Sept. 1921 to June 1922) and Univ. of Kans. (Sept. 1923 to June 1924). *TT 1: P 1.*—April to July 1918 Head Chainman, F.A.P. 42, Pawhuska to Pershing, Okla. Sept. 1919 to Sept. 1920 Draftsman, Midland Tool and Supply Co., Pawhuska, on oil-well tool designing and detailing. *TT 0.4: SP 0.4.*—July 1924 to March 1925 Chf. of Party, Republic of Mexico, Mineral Land Sur-

veyor for Dist. of Rayon, Chihuahua. *TT 0.7: P 0.7: RC 0.7.*—April 1925 to May 1929 with County Engr., Osage County, Okla., until Jan. 1926 as Draftsman, estimating, detailing and general layout, earthwork, bridge and alignment plans of county, state and U. S. highways, then Chf. of Party, on location surveying, classification, estimating and construction of roads, bridges and drainage structures on county highways. *TT 4.1: SP 0.2: P 3.9: RC 3.4: D 1.*—June 1929 to date with Oklahoma State Highway Comm., until Oct. 1930 as Chf. of Party, on construction of reinforced concrete paving (approx. 27 miles), setting form hubs, laying grade and calculating earth quantities, grade and drainage in Lincoln County (4½ miles) and Oklahoma County (1 mile), being in responsible charge of field work and calculating quantities; Nov. 1930 to July 1931 Chf. of Party in complete charge on reconnaissance and final location of State Highway 37, Norman, Okla., to Cleveland Co. line east and in complete charge of field work on approx. nine miles of reinforced concrete paving, F.A.P. 131; Aug. to Sept. 1931 Instrumentman on location; at present Chf. of Party on construction. *TT 2.3: SP 0.1: P 2.2: RC 2.2.*—*TT 8.5: SP 0.7: P 7.8: RC 6.2: D 1.* Refers to M. E. Binckley, H. W. Crawford, C. W. McFerron, G. G. Toler, G. Whittenberg.

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(9) **McDONNELL, PORTER WILSON,** Jun., 321 Valentine Bldg., Toledo, Ohio. (Elected June 4, 1928.) (Age 28. Born Titusville, Pa.) 1920 to 1922 and Sept. 1923 to Feb. 1924 student, Univ. of Mich. *TT 1: P 1.*—Summer 1922 Eng. Camp Davis, Univ. of Michigan.—Sept. 1922 to Sept. 1923 Inspector, Rodman, Instrumentman, Draftsman and Res. Engr., Michigan State Highway Dept. *TT 0.5: SP 0.5.*—Feb. to May 1924 Chf. of Survey Party with W. E. Brown, Civ. and Landscape Engr., Miami Beach, Fla. *TT 0.1: SP 0.1.*—May to Aug. 1924 Instrumentman, Washtenaw County (Mich.) Road Comm. *TT 0.2: SP 0.2.*—Feb. to Oct. 1925 Chf. of Survey Party with Fred L. Kuebler, Civ. Engr. and Surveyor, Toledo, Ohio. *TT 0.7: P 0.7: RC 0.7.*—May to July 1926 Draftsman, Erie Office, Pennsylvania State Highway Dept. *TT 0.1: SP 0.1.*—July to Dec. 1926 Instrumentman with B. E. Briggs, Civ. Engr., Erie, Pa. *TT 0.2: SP 0.2.*—Oct. 1925 to May 1926 Draftsman, and Jan. 1927 to March 1928 Senior Engr., City Engrs. Office, Toledo, Ohio. *TT 1.5: SP 0.3: P 1.2: RC 1.2.*—March 1928 to March 1929 Engr. for M. Rabbitt & Sons, Inc., Marine and Highway Contrs., Toledo, Ohio *TT 1: P 1: RC 1.*—March to Sept. 1929 Engr. with Geo. Champe & Associates, Cons. Engrs., Toledo, Ohio. *TT 0.5: P 0.5: RC 0.5.*—Sept. 1929 to April 1931 Field Engr. with Waddell & Hardesty, New York City, on high level suspension bridge at Toledo, Ohio. *TT 1.6: P 1.6.*—April 1931 to date Pres. and Mgr., Toledo Surveyor, Inc., surveying, mapping and subdivision. *TT 0.7: P 0.7: RC 0.7.*—*TT 8.1: SP 1.4: P 6.7: RC 4.1.* Refers to G. Champe, A. S. Forster, H. P. Jones, L. T. Owen, R. H. Randall, G. N. Schoonmaker, A. H. Smith, G. A. Taylor.

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(3) **OWEN, WILLIAM VICTOR,** Jun., 158 State St., Albany, N. Y. (Elected Oct. 1, 1926.) (Age 28. Born Buffalo, N. Y.) 1926 B. S. in Eng., Univ. of Mich. *TT 4: P 4.*—June 1926 to April 1927 Structural Draftsman and Detailer, Turner Constr. Co., Gen. Contrs., Buffalo, N. Y., drafting and on design of concrete details, reinforcing, etc., also 2 months in field supervising construction of pile foundation. *TT 0.4: SP 0.4.*—April 1927 to July 1929 Chf. Inspector of Eng. Improvements, Town of Tonawanda, N. Y., in charge of inspection and acceptance of all public contract work, including highways, water supply, sewers, sewage-disposal plant, refuse incinerators, park improvements, etc. *TT 2.2: P 2.2: RC 2.2.*—July 1929 to date Asst. Engr., Chf. Engr.'s Office, New York Telephone Co., Albany, N. Y., on structural design of telephone buildings, garages, storehouses, strengthening of existing buildings, investigating foundation conditions, water pressure and on design of retaining walls, also supervising construction in field. *TT 2: SP 0.5: P 1.5: RC 1.5: D 1.5.*—*TT 8.6: SP 0.9: P 7.7: RC 3.7: D 1.5.* Refers to B. L. Cushing, G. C. Diehl, F. O. Francis, E. P. Lupfer, H. Ryon, P. H. Woodworth.

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(1) **POWELL, JOHN EDWARD CHARLES,** Jun., 7441 Sixty-second St., Glendale, N. Y. (Elected Jan. 18, 1926.) (Age 27. Born Brooklyn, N. Y.) 1925 C. E., Rens. Pol. Inst. *TT 4: P 4.*—July 1925 to April 1926 Eng. Asst. with H. S. Swan, City Planner, New York City, city planning surveys, designs and reports covering zoning, traffic-congestion studies, grade-crossing elimination, thoroughfare and civic-development plans. *TT 0.8: P 0.8.*—April 1926 to May 1927 Draftsman with Gibbs & Hill, Cons. Engrs., New York City, design of steel bents and columns, foundations, guys, guy anchors and catenary trolley-wire system for railroad electrification. *TT 1.1: P 1.1: RC 0.7.*—May 1927 to Sept. 1929 Eng. Draftsman, Designer and Detailer, Asst. Engr., New York Central R. R.,

New York City, design of steel railroad and highway bridges, plain and reinforced foundations, design of falsework and investigating and strengthening designs for timber trestles, design of timber framing, floors and walls of ferry house building. *TT 2.3: P 2.3: RC 1.3.*—Sept. 1929 to Dec. 1930 Asst. Engr. with Frederic R. Harris, Cons. Engr., New York City, design and studies of piers, dry docks, temporary subway construction, heavy foundations and handling cranes. *TT 1.2: P 1.2: RC 1.2.*—April 1931 to date in charge of design squad, Gibbs & Hill, Cons. Engrs., New York City, supervising design of steel bents and columns, foundations, guys and guy anchors for railroad electrification (catenary). *TT 0.8: P 0.8: RC 0.8*—*TT 10.2: P 10.2: RC 4.* Refers to R. F. Bessey, W. S. Diver, F. R. Harris, A. F. Lipari, J. E. Porter, W. L. Unger.

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(1) **VELZ, CLARENCE JOSEPH**, Jun., 2411 Hudson Terrace, Coytesville, N. J. (Elected Oct. 1, 1926.) (Age 30. Born Shakopee, Minn.) 1924 B. S. in C. E., and 1929 C. E., Univ. of Minn. *TT 4: P 4.*—2 years graduate study, Public Health Eng., Columbia Univ. —1918 to 1920 First Asst. to Donald Childs, Minnesota Highway Dept., on location and construction, also municipal work; 1920 to 1923 (while student) plans and surveys for 27 miles Federal Aid Highways. *TT 1: SP 1.*—June to Nov. 1924 Leveler, New York Central R. R. Co. *TT 0.2: SP 0.2.*—Nov. 1924 to Aug. 1925 Rodman, Illinois Central R. R. Co. *TT 0.4: SP 0.4.*—Aug. 1925 to April 1926 Chf. Engr., C. A. Blair & Co., Inc., in charge of development (4300 acres) in Florida, mapping, paving, water supply, sewage disposal, designing and supervising construction. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—April 1926 to date Chf. Engr. with F. J. Oleri, Mun. and San. Consultant, on investigations, reports and design, sewerage systems, pumping station, disposal works, etc., precise surveying, supervising construction of several large school buildings and commercial structures, design and supervision of construction of parks, playgrounds, swimming pools and reinforced concrete stadium. *TT 5.7: P 5.7: RC 5.7: D 5.7.*—*TT 12: SP 1.6: P 10.4: RC 6.4: D 6.4.* Refers to F. Bass, G. V. Guerin, Jr., H. T. Larsen, J. Muss, F. J. Oleri.

The Board of Direction will consider the applications in this list not less than thirty days after the date of its issue.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

BOARD OF DIRECTION MEETINGS

January 18-19, 1932:

A Quarterly Meeting will be held at New York, N. Y.

ANNUAL MEETING, NEW YORK, N. Y.

January 20, 21, and 22, 1932

January 20, 1932:

Morning. — Annual Meeting. Conferring of **Honorary Membership, and Presentation of Medals and Prizes.**

Afternoon. — Technical Meeting.

Evening. — President's and Honorary Members' Reception and Dinner Dance.

January 21, 1932:

Morning. — Technical Division Sessions.

Afternoon. — Technical Division Sessions.

Evening. — Entertainment and Smoker.

January 22, 1932:

All-Day Excursion.

The Reading Room of the Society is open from 9:00 A. M. to 5:00 P. M. every day, except Sundays and holidays; from May to September, inclusive, it is closed on Saturdays at 12:00 M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it is a place for meeting others. There is a file of 274 current periodicals, the latest technical books, and the room is well supplied with writing tables.